Non-Concurrence Process Record for NCP-2011-014

The U.S. Nuclear Regulatory Commission (NRC) strives to establish and maintain an environment that encourages all employees to promptly raise concerns and differing views without fear of reprisal and to promote methods for raising concerns that will enhance a strong safety culture and support the agency's mission.

Individuals are expected to discuss their views and concerns with their immediate supervisors on a regular, ongoing basis. If informal discussions do not resolve concerns, individuals have various mechanisms for expressing and having their concerns and differing views heard and considered by management.

Management Directive MD 10.158, "NRC Non-Concurrence Process," describes the Non-Concurrence Process (NCP). http://pbadupws.nrc.gov/docs/ML0706/ML070660506.pdf

The NCP allows employees to document their differing views and concerns early in the decision-making process, have them responded to, and attach them to proposed documents moving through the management approval chain.

NRC Form 757, Non-Concurrence Process is used to document the process.

Section A of the form includes the personal opinions, views, and concerns of an NRC employee.

Section B of the form includes the personal opinions and views of the NRC employee's immediate supervisor.

Section C of the form includes the agency's evaluation of the concerns and the agency's final position and outcome.

NOTE: Content in Sections A and B reflects personal opinions and views and does not represent official factual representation of the issues, nor official rationale for the agency decision. Section C includes the agency's official position on the facts, issues, and rationale for the final decision.

The agency's official position (i.e., the document that was the subject of the non-concurrence) is included in ADAMS accession number ML12298A415.

NRC FORM 757
NRC MD 10.158
(3-2009)

U.S. NUCLEAR REGULATORY COMMISSION

NON-CONCURRENCE PROCESS

SECTION A - TO BE COMPLETED BY NON-CONCURRING INDIVIDUAL

TITLE OF DOCUMENT **South Texas Project Combined License Application Review: SER with No Open Item Chapter 2.4**	ADAMS ACCESSION NO. **ML 111450749**
DOCUMENT SPONSOR **Nilesh Chokshi, Deputy Director, NRO/DSER**	SPONSOR PHONE NO. **301-415-1634**
NAME OF NON-CONCURRING INDIVIDUAL **Hosung Ahn**	PHONE NO. **301-415-1398**

☑ DOCUMENT AUTHOR ☐ DOCUMENT CONTRIBUTOR ☐ DOCUMENT REVIEWER ☐ ON CONCURRENCE

TITLE **Hydrologist**	ORGANIZATION **NRO/DSER/RHEB**

REASONS FOR NON-CONCURRENCE

See attachments:

1) Attachment 1: Reasons for Non-Concurrence (ML12285A389)

☐ CONTINUED IN SECTION D

SIGNATURE Hosung Ahn	DATE 6-8-2011

SUBMIT FORM TO DOCUMENT SPONSOR AND COPY TO YOUR IMMEDIATE SUPERVISOR AND DIFFERING VIEWS PROGRAM MANAGER

Attachment 1: Reasons for Non-Concurrence

Title: South Texas Project Combined License Application Review: SER with no Open Item Chapter 2.4
Name: Hosung Ahn
Date: June 8, 2011

Summary

This document addresses three non-concurring issues on the Safety Evaluation Report (SER) for the South Texas Project (STP) Combined License Application (COLA). The non-concurring individual, Dr. Hosung Ahn, is hydrology lead for reviewing the STP COLA FSAR Chapter 2.4 Hydrology. I am filing this non-concurrence package as a SER document contributor who is not on the list of document concurrences (~~ML1114507490~~ ML111450749 for concurrence sheet and ~~ML1114500734~~ ML111450734 for SER).

My concerns are related to determining two site parameters - design basis flood level and maximum groundwater level - which are important site parameters for structural design and flood protection of safety-related facilities. It is my opinion that the staff's review of these two parameters was not properly done. Specifically, the postulated main cooling reservoir breach scenario and design basis flood level estimation in the SER are neither accurate nor conservative. To justify my technical position and to provide a potential solution, I performed a re-analysis of design basis flood. The full re-analysis report will be provided as Enclosure 1 to this package.

The applicable regulations in estimating these two site parameters are: (a) 10 CFR 52.79(a)(1)(iii) and GDC 2 of Part 50, as they relate to identifying hydrologic site characteristics with appropriate consideration of **the most severe** of the natural phenomena; and (b) 10 CFR 100.20(c)(3), as it requires to identifying and characterizing the **maximum probable** hydrologic events such as wind speed and precipitation for site safety analysis. The following statements describe detailed concerns on each issue, followed by a proposed solution.

Issue #1: Main Cooling Reservoir (MCR) Breach Flood Analysis in SER Section 2.4.4

1) ABWR DCD Tier 1 requires that the design basis flood level be less than 1 ft below the plant grade. STP identified that a postulated MCR levee breach scenario will create design basis flood that exceeds the DCD limit – It is handled as a departure in FSAR. To formulate a breach scenario, STP estimated width and time of levee breach based using two empirical regression equations: Froelich equation for breach width and MacDonald and Langridge-Monopolis (MLM) equation for breach time. In my opinion, STP's approach is incorrect because (a) it is a mix-and-match approach (use two different equations for one breach scenario) which is not acceptable technically; (b) The Froehlich breach width equation,

which is good for the breach widths less than 164 ft, is not applicable to the MCR breach which is much longer than 164 ft in length; and (c) STP excluded the MLM breach erosion volume equation that may result in the most conservative breach width. Especially, STP justified in a RAI response that the MLM equation is not for breach width. It is my opinion that the STP's justification is incorrect, because the MLM breach erosion volume equation that can be used to estimate breach width (e.g., dividing the erosion volume by levee cross section area) has been classified and used widely for estimating breach width. My re-analysis report addresses this issue.

2) The Pacific Northwest National Laboratory (PNNL) wrote a draft SER. To postulate MCR breach, PNNL simulated breach outflow hydrographs using the BREACH model. However, PNNL set up the BREACH model incorrectly: They set up the BREACH model to limit levee width of 1000 ft and tailwater bottom width of 600 ft so that the simulated breach width and breach outflows are constricted by unrealistically small levee and channel cross sections. This constriction results in a less conservative design basis flood level. Enclosure 2 shows the corrected result of from my analysis. My re-analysis report presents a comprehensive MCR breach analysis considering the above correction as well as the effects of scouring hole, cement blocks, and levee's sand core on the breach process (Enclosure 1). The PNNL modeling has serious deficiencies, so it cannot be accepted as a credible safety analysis.

3) PNNL simulated the BREACH model with three surface roughness coefficient values (n-value of 0.025, 0.05, 0.075), then picked the case of 0.075 as a postulated breach scenario. However, PNNL never justify the selection of the n-value of 0.075. My re-analysis shows that n-value could be higher than 0.075.

4) PNNL confirmed the postulated breach scenario using an actual breach case from the Martin Cooling Pond in Florida, but this process is highly subjective compared to the statistical regression methods that used multiple historical data points to come up the best-fitting (or bounding) regression line of breach parameters. They also estimated breach parameters using selected empirical equations; however they do not provide the reasons why the conservative breach parameter estimates using empirical equations were ignored in SER.

5) The BREACH model is based on the Meyer-Peter & Muller (MPM) sedimentation equation. However, many hydrologists have noted that the MPM method is not adequate for dam breach conditions as it is developed for estimating sedimentation in small mountain areas. The MCR breach condition is quite different from the mountain condition in terms of slope and bottom condition. Because the result of BREACH model is highly depending on input parameters, many researchers discredit the BREACH model. Even the author of the BREACH model recommended using the BREACH model as a supplemental purpose only. PNNL's approach to estimate breach parameters using the BREACH model results in a biased and less conservative breach scenario, as my analysis demonstrates.

6) PNNL's BREACH simulations did not consider the effects of the scouring of the foundation during the MCR breach process. Proper consideration of the scouring effects will substantially increase the breach outflows and the resulting flood level at the site as the attached re-analysis report shown. In summary, the postulated MCR breach scenario and the resulting DBF flood level estimates in SER are inaccurate and non-conservative.

Issue #2: Flood Analysis of Hurricane and MCR Breach Combination in SER Section 2.4.5

1) At the beginning of the FSAR review, the staff noted that combined events such as a combination of storm surge and MCR breach, could create a more severe flood than that of a MCR breach only. In this regard, PNNL reviewed the applicant's storm surge modeling and concluded that the applicant's probable maximum hurricane (PMH) used in the surge model is conservative. It is my opinion that the STP's PMH parameters are not conservative because the applicant's PMH is based on the NOAA's manual (NWS-23) which was developed in 1979 but has not been updated for the storms observed during the past 30 years. It is generally known that the intensity and frequency of hurricanes have been increased during the past decades. SER does not address this change adequately.

2) PNNL concluded in SER Section 2.4.5 (p. 81) that the STP's ADCIRC simulations for determining the storm surge level at the site is adequate. However, they provide no background information or any justifications on that conclusion. PNNL never review the input and output of the STP's ADCIRC model nor conduct any independent confirmatory modeling other than SLOSH simulations which were discarded by the applicant. The main reason for discarding the SLOSH model is that the STP's SLOSH model is less accurate than STP's ADCIRC model that uses finer grids with detailed topography data. Since STP adopted to use the ADCIRC model, NRC should review the STP's ADCIRC model to support the conclusion of adequacy of surge modeling and to justify the reasons why the combined event of hurricane and MCR breach is discarded for consideration.

Issue #3: Maximum Groundwater Level in SER Section 2.4.12

1) ABWR DCD Tier 1 specifies the requirement of the maximum groundwater level below two feet plant grade. This DCD parameter is used in structural design and the NRC reviewer must check whether the site groundwater levels exceed the DCD limit or not. The STP FSAR adopts a site characteristic for a maximum groundwater level of 28 ft MSL under the proposed plant grade of 34 ft MSL. STP proposed to install a 2-ft clay layer or cap immediately below grade level to interrupt infiltration from surface ponding to groundwater body through backfill materials and to maintain the maximum groundwater level. STP also proposed a stone layer to prevent erosion of the clay layer from MCR breach flood, so that there is no interaction between surface and ground waters to meet the DCD condition.

2) However, the NRC staff assumed conservatively that the clay cap could be eroded away during the MCR breach flood, resulting in a saturation condition of the entire vertical profile from the plant grade to the level of 28 ft MSL. This condition creates a departure of the DCD condition, which is different from what STP assumed in the FSAR. The NRC SER states that this groundwater condition is included in the engineering evaluation as discussed in SER Section 3.8, but it is not confirmed nor committed yet. It is author's opinion that the

SER should describe why the stone layer is breakable and that FSAR should state this departure condition, including properly reflecting the departure condition in structural designs and flood protection measures. So far, staff has not informed this departure condition to STP. It is my opinion that the process of reviewing and handling the maximum groundwater level in SER is inadequate for the above reasons. The saturation condition should be handled as a DCD departure.

Proposed Solution

It is my opinion that both FSAR and SER contain serious errors in the MCR breach flood analysis. The analysis is also not conservative to meet the requirements of NRC regulation as my attached re-analysis report demonstrated. The result of my re-analysis shows that the design basis flood level could be much higher than the STP's estimate. Therefore, I am recommending with this non-concurrence process that SER should be revised before presenting it to the ACRS and ASLB.

Enclosure 1: Re-analysis of MCR Breach Flood Analysis (ML12285A282)
Enclosure 2: PNNL's Calpack – commented and corrected version by Hosung Ahn (ML12285A292)

Enclosure 1: Re-Analysis of MCR Breach Flood

(Attachment to the Non-concurrence on the Safety Evaluation Report Section 2.4.4 for the South Texas Project Units 3&4 Combined License Application)

June 20, 2011

Hosung Ahn, Ph.D., P.E.

U.S. Nuclear Regulatory Commission

NRO/DSER/RHEB

Table of Contents

1 Executive Summary

This document addresses one of the three non-concurring issues raised by Dr. Hosung Ahn on the Safety Evaluation Report (SER) Chapter 2.4 for the South Texas Project (STP) Combined License Application (COLA). This report address the issue related to determining a site parameter - design basis flood level - which is critical for structural design and flood protection of safety-related facilities.

STP provided in their Final Safety Analysis Report (FSAR) the estimation of the design basis flood (DBF) level caused by a postulated main cooling reservoir (MCR) breach. NRC staff reviewed the estimation of the DBF elevation and concluded in the SER that the applicant's DBF estimation is acceptable. PNNL help the NRC staff in reviewing the STP's MCR breach analysis. PNNL performed a breach modeling as a confirmatory analysis, however their breach modeling includes a serious error. It is my opinion that the postulated main cooling reservoir breach scenario and design basis flood level estimation in both FSAR and SER are neither accurate nor conservative. To justify my technical position and to provide a potential solution, I performed a re-analysis of the MCR breach flood.

The applicable regulations in estimating the design basis flood level are: (a) 10 CFR 52.79(a)(1)(iii) and GDC 2 of Part 50, as they relate to identifying hydrologic site characteristics with appropriate consideration of **the most severe** of the natural phenomena; and (b) 10 CFR 100.20(c)(3), as it requires to identifying and characterizing the **maximum probable** hydrologic events such as wind speed and precipitation for site safety analysis.

This report includes all available relevant information on the MCR breach analysis and modeling. The analysis done in this report is very comprehensive – It is a multiple-step process including the following modeling:

- Levee breach parameter estimation using all available empirical equations
- Validation of breach parameters and outflows using the BREACH model
- Simulating breach outflow using the FLDWAV model, and
- Routing of breach flood at the site using the FLO-2D model.

The result of the breach and flood modeling indicates that the reasonably conservative DBF levels at the proposed Units 3 & 4 would be about 45 ft MSL considering a enough margin for the wind wave setup and modeling uncertainty. This new estimate is much higher than the STP's DBF estimation of 40 ft MSL. As this increase in flood level is significant in structural design and flood protection measures, the author of this report strongly recommend revising both the FSAR and the current version of the SER.

In addition to site specific data, this report includes a lot of useful information in relation to the levee breach flood analysis in general. However, the readerships of this report should note that the content of this report could be applied to other levee breach flood analyses but not directly used because the levee breach flooding is governed by many site-specific parameters and conditions.

2 Introduction

Final Safety Analysis Report (FSAR) Section 2.4.4 of the South Texas Project (STP) combined licensing application (COLA) addresses potential dam failures. The purpose of the dam failure analysis in FSAR is to ensure that potential external flood hazard to safety-related structures due to failure of dam is considered in the plant design and flood protection measures. This FSAR section covers postulation of dam failure scenarios, hydrodynamic analysis of dam failures, and determination of flood characteristics (e.g., flood level, duration, velocity, etc.) at the site. The dam failure flood analysis used in FSAR 2.4.4 is a deterministic approach – We assume a dam failure without considering the condition of the dam or probabilities of failure occurrence and consequence.

The dam failure analysis in the STP's FSAR includes the potential main cooling reservoir (MCR) embankment breach and the effects of the breach on the site flooding. STP identified that the MCR breach is the controlling design basis flood (DBF) scenario among all potential flood events, including rainfall, hurricane, tsunami, and dam breaches. They also identified that the controlling flood level is higher than the bounding design flood level of one feet below the plant grade specified in the ABWR Design Certificate Document (DCD), thus handled the DBF level as a DCD departure. The ABWR DCD requires that the maximum flood level should be 1 ft below the plant grade.

The MCR covers an area of approximately 3 miles by 4 square miles and is formed by an embankment levee about 12.4 miles long. The area of the MCR is about 7,000 acres. For the Units 3&4, STP proposed to use the MCR which was originally built for four reactor units. The MCR was designed for the normal maximum operation level of 49 ft mean sea level (MSL). However, STP has been operating the reservoir at a pool level of 45 ft MSL after a field seepage test in early 1990s. With the proposed two units, STP proposed to raise the operating reservoir level from 45 ft MSL to 49 ft MSL. The raise of the operating level could increase the potential of MCR levee breach which is a significant safety issue.

The STP's estimation of the DBF elevation caused by a postulated MCR breach scenario is 40 ft above MSL. The breach scenario is based on the estimated breach width of 417 ft and breach time of 1.7 hours.

The NRC staff assisted by the PNNL completed the review of the STP's MCR breach analysis in May 2011, and concluded that the applicant's analysis of the MCR breach flood is acceptable (refer to the SER Section 2.4.4). However, the author of this report (Dr. Hosung Ahn) found that the analyses of MCR breach flood by both STP and PNNL are inaccurate and non-conservative. Therefore, the author made a re-analysis of the MCR breach flood in a conservative manner. This report summarizes the result of the re-analysis and findings. Section 3 of this report summarizes the regulatory requirement of dam failure analysis for reactor licensing. Sections 4 and 5 discuss the problems in STP's and PNNL's dam breach analyses, followed by the re-analysis of MCR breach flood in Section 6.

3 Regulatory Basis

The applicable regulatory requirements for analyzing the effects of dam failures on the proposed new reactors are as follows:

- 10 CFR Part 100, as it relates to identifying and evaluating hydrological features of the site. In site evaluation, 10 CFR 100.20(c) specifies the requirement to consider physical site characteristics, including seismology, meteorology, geology, and hydrology.

- 10 CFR 100.23(d), as it sets forth the criteria to determine the siting factors for plant design bases with respect to seismically induced floods and water waves at the site.

- 10 CFR 52.79(a)(1)(iii) and GDC 2 of Part 50, as it relates to identifying hydrologic site characteristics with appropriate consideration of **the most severe** of the natural phenomena that have been historically reported for the site and surrounding area and with **sufficient margin** for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

In addition, the NRC staff has been used appropriate regulatory positions of the following regulatory guides for the acceptance criteria of dam failure analyses:

- RG 1.27, "Ultimate Heat Sink for Nuclear Power Plants"

- RG 1.59, "Design Basis Floods for Nuclear Power Plants," as supplemented by best current practices

- RG 1.102, "Flood Protection for Nuclear Power Plants."

Specifically, Part 100.20(c)(3) states that the **maximum probable** wind speed and precipitation must be identified and characterized in the analysis of reactor site safety. Correspondingly, the Regulatory Position 1.c in RG 1.59 specifies the requirement of analyzing dam failures to identify **the worst** site-related flood at a nuclear power plant. Section 2.4.4 of NUREG–0800 (SRP) provides technical rationale and acceptance criteria for the analysis of dam failures. In this SRP section, the terms 'conservative' is used 8 times and the term 'the most severe' is appeared 7 times in order to emphasis the conservatism that must be applied to the analysis of dam failures. The author of this report claims that the dam failure analyses by both STP and PNNL do not meet these regulatory requirements.

4 MCR Breach Analysis Submitted by STP

STP presented the main cooling reservoir (MCR) breach flood analysis in the FSAR Section 2.4.4 and the responses of relevant RAIs. The MCR is a manmade, in-ground reservoir enclosed by about 12.4 miles embankment levee. The function of the MCR is to supply water to dissipate the heat generated from the nuclear power plants. STP analyzed the onsite flood hazard resulting from a postulated breach of the north MCR embankment levee located about 2,300 ft south from the proposed Units 3 and 4.

The MCR embankment levee was built with 40-feet-high rolled soil materials above the clay-sand-silt top surface. The interior face (reservoir side) of the dyke is lined with 2 ft thick soil cement blocks to absorb wave actions. The outside face of the levee is covered by grass on bare soils (refer to Figure 4-1). The design normal maximum operating water level in the MCR is 49 ft MSL. STP postulated the MCR dyke failures caused by the excessive seepage from: (1) piping through the foundations of embankment, (2) seismic activity leading to liquefaction of the foundation soils, and (3) erosion of the embankment due to overtopping or wind-wave events.

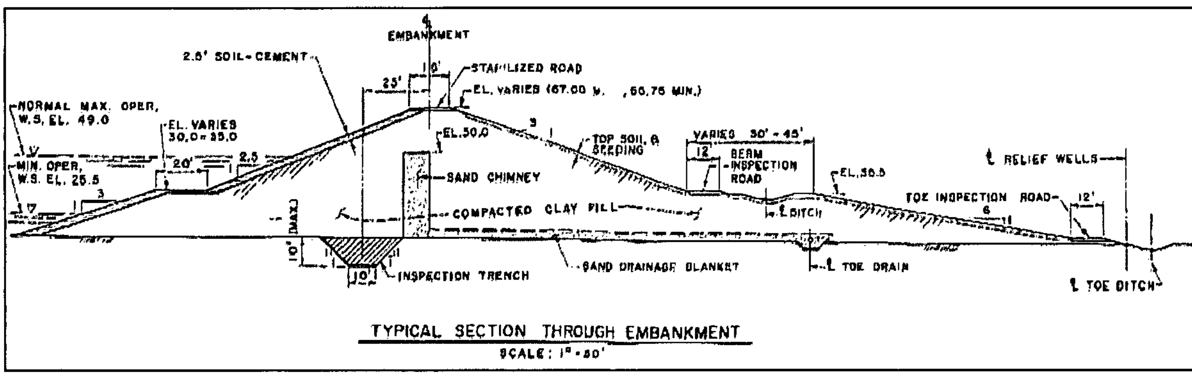


Figure 4-1. Cross section of the MCR embankment levee (provided by STP)

STP postulated and analyzed a northern MCR levee breach that results in the most severe flood (in terms of the flood level) at the proposed site. The following summarizes the STP's MCR breach flood analysis:

(1) STP estimated levee breach parameters (e.g., width and time of breach) based on the two empirical equations: Froelich equation for breach width and MacDonald and Langridge-Monopolis (MLM) equation for breach time. STP validated these parameter estimates using the NWS BREACH model (Fread, 1991; or just BREACH hereafter).

(2) STP simulated the MCR breach outflow hydrograph using the FLDWAV model (Fread and Lewis, 1998) with the above estimated breach parameters.

(3) STP estimates the flood characteristics (e.g., flood level, time, velocity, etc) at the plant sites using the RMA2 model (Donnell et al., 2008) with the FLDWAV breach outflow hydrograph.

Specifically, STP used empirical equations to estimate dam breach parameters for earth-filled structures as recommended by the U.S. Bureau of Reclamation (USBR) (Wahl 1998). STP assumed that a service road immediately downstream of the toe of the MCR dyke will be the control bottom (29 ft MSL) for the postulated breach.

STP used the Froehlich equation to estimate the breach width. The estimated average breach width (e.g., top and bottom of the breach) is 417 ft. The STP estimated the time to failure of 1.7 hours using the MLM bounding equation. STP obtained the peak breach outflow of 62.600 cfs using the NWS BREACH model. They also estimated the peak breach outflow rate of 130,000 cfs using the FLDWAV model, which was used in the RMA2 model.

Evaluation of STP's Breach Analysis

It is the author's opinion that STP's breach analysis is not conservative in terms of its approach and model parameters. To the author, breach parameter estimates are of concerns as they are very sensitive to the resulting DBF level. The following bullets summarize this issue:

- STP used **a mix-and-match approach** in estimating MCR breach parameters – The MLM equation for time to peak and the Froehlich equation for breach width. This results in less conservative parameters.

- In estimating the MCR breach width, STP ignored the MacDonald and Langridge-Monopolis (MLM) equation (USBR, 1998) which results in the most wide breach width. In a RAI response, STP stated that the MLM equation is not for breach width. This statement is incorrect. The MLM equation which was originally developed for estimating breach eroding volume has been classified and used widely in estimating breach widths (for examples, Table 1 in Wahl (1997); Wahl (2003); Gee (2010); Brunner (2007); and many others).

- STP used the Froehlich's breach width equation (1995), however USBR (1998) states that the Froehlich's equation appears to be the best predictor for the breach widths less than 164 ft because the breach width records used to develop the Froehlich equation are small. In other words, Froehlich's breach width equation is not applicable to the MCR breach case.

- The breach parameters estimated by the BREACH model is very sensitive to the roughness condition (so called the Manning's n-value) of the breach bottom. The Manning's n-value of 0.045 used in STP's BREACH modeling is not conservative. Also, STP does not consider the erosion of the levee below the 29 ft MSL in their breach analysis. Considering these two factors will substantially increase breach width and outflows as demonstrated in Section 6 of this report.

- The set-up of both STP's and PNNL's BREACH model is incorrect. In reality, the downstream tailwater condition of the MCR breach is wide 2-dimentional overland flow. However, both STP and PNNL set a limited tailwater cross-section width of 600 ft and levee crest width of 1000 ft, which restricts the simulation of breach width and outflows. This constriction results in an underestimation of the DBF flood level – It will be discuss further in Section 6 of this report.

5 MCR Breach Analysis by PNNL

PNNL performed an independent confirmatory analysis of the MCR breach flood using the breach models developed by the applicant. The following summarizes the procedure used by PNNL:

(1) PNNL simulated breach widths and outflow hydrographs using the BREACH model with three Manning's n-values, then picked the most conservative one as a postulated breach scenario.

(2) They confirmed the breach scenario using empirical breach parameter equations as well as an actual breach case from the Martin Cooling Pond in Florida.

(3) PNNL used the applicant-provided RMA2 model to simulate flood characters at the site.

PNNL carried out a sensitivity analysis by varying some of the BREACH model parameters to determine the sensitivity of model input parameters (e.g., levee geometry, embankment material properties, headwater/tailwater conditions, etc.) on the breach parameters (e.g., width, time, peak flow) (PNNL, 2011). As a result of the BREACH sensitivity analysis, PNNL identified that the Manning's n-value that represents the bottom roughness condition is very sensitive to the breach parameters and outflows. They used the following three different n-values in the BREACH simulation:

- Simulation 1: n = 0.025
- Simulation 2: n = 0.050
- Simulation 3: n = 0.075.

Then, PNNL selected the Simulation 3 as the postulated breach scenario as it produces the largest breach peak outflow (130,000 cfs). The corresponding peak breach width and time are 515 ft and 1.99 hours.

PNNL also compared the BREACH-simulated outflow characteristics (peak flow and time, breach width) with the selected historical data from the database prepared by USBR (1998). From the review of the recorded breach events, PNNL selected a recorded breach event occurred in 1979 at the Martin Cooling Pond in Florida. The reservoir condition is similar to that of the MCR in terms of storage volume and water depth. The final average breach widths for the Martin Cooling Pond breach and the estimated MCR breach are 607 ft and 688 ft, respectively. Based on this comparison, PNNL concluded that their MCR breach parameters and flood estimates are reasonable.

Evaluation of PNNL's Breach Analysis

However, PNNL does not justify the selection of the n-value of 0.075 or why this value is conservative. Further, the author of this study found that PNNL's BREACH model to estimate the breach outflows and the sensitivity analysis is flawed by an incorrect model set-up. In

addition, validating the BREACH simulating result by a single arbitrarily selected breach record is not objective. The following summarizes the author's main concerns on the PNNL's analysis:

- PNNL's calculation package (2011: calpack) provides breach parameter estimates using three empirical equations (namely USBR, MLM, Froehlich) and the BREACH model. However, PNNL does not explain why MLM estimates that are the most conservative are excluded in further breach analysis. This is against general engineering practice or the guidance by federal agencies. Both USBR (1998) and USACE (2007) recommend using all available methods to determine breach parameters as the process involves a lot of uncertainty.

- PNNL used a faulty BREACH model setup by using a limited levee crest width of 1000 ft and tailwater bottom width of 600 ft (refer to Figure 5-1), which constrict the breach process and the resulting breach outflows. Finally, this modeling error results in an underestimation of breach width, outflows, and DBF flood level. This is an unacceptable modeling error. *(Note: The author notified this error to PNNL and NRC staffs in advance (February, 2011) of completing the SER, but my input has been ignored)*.

- The NWS BREACH model is based on the Meyer-Peter & Muller (MPM) sedimentation equation which results in a lowest sedimentation rate compared to other methods (refer to "Erosion and Sedimentation", by P. Julien, 2010). Many hydrologists pointed out that the MPM method is not adequate for dam breach conditions as it is developed for small mountain creeks.

- Because the result of BREACH model is highly depending on the model inputs, many hydrologists discredited the use of the BREACH model (e.g., Wahl, 1999). Also, the author (Dr. Fread) of the BREACH model recommends that BREACH model be used supplemental purpose only. PNNL's approach that relies only on the BREACH could be biased as this report demonstrated.

- Both STP and PNNL ignored the effects of sand core and clay cement blocks on the BREACH mode. Also they ignored the effects of scour hole on the breach process. Proper consideration of these factors will increase breach outflow and DBF level estimates substantially this report will be demonstrated.

The next section presents a re-analysis of MCR breach flood by considering the above concerns and by adding a reasonable conservatism on the estimation.

(a) PNNL's BREACH Input File for Simulation 3

```
MCR
50.9    66.     29.     32.     0.      0.      .02     .8
30.     30.     30.     0.      0.      0.      0.      0.
0.      1.      100.    0.      0.      0.      0.      0.
7050.   7000.   6850.   5690.   2050.   0.      0.      0.
52.     49.     29.     25.5    20.     0.      0.      0.
28.     29.     32.     34.     40.     45.     50.     0.
0.      600.    1200.   1600.   2000.   2400.   2800.   0.
.06     .06     .06     .06     .06     .06     .06     0.
2.5     3.      0.      0.      0.      0.      0.
0.      0.      0.      0.      0.      0.      0.
0.001   .35     105.    .075    15.     200.    8.
1.      16.     1000.   8.      0.      0.      0.      0.
.001    0.001   .1      120.    .01     1.      0.
```

This small bottom tailwater section (600 ft) constricts the simulated breach width and outflows.

Again, this small levee crest length constricts the breach width and outflows.

Manning's n-value

(b) Hosung Ahn's BREACH Input File for Simulation 3

```
MCR Ahn's Run
50.9    66.     29.     32.     0.      0.      .02     .8
30.     30.     30.     0.      0.      0.      0.      0.
0.      1.      100.    0.      0.      0.      0.      0.
7050.   7000.   6850.   5690.   2050.   0.      0.      0.
52.     49.     29.     25.5    20.     0.      0.      0.
28.     29.     32.     34.     40.     45.     50.     0.
0.      3000.   3000.   3000.   3000.   3000.   3000.   0.
.06     .06     .06     .06     .06     .06     .06     0.
2.5     3.      0.      0.      0.      0.      0.
0.      0.      0.      0.      0.      0.      0.
0.001   .35     105.    .075    15.     200.    8.
1.      16.     3000.   8.      0.      0.      0.      0.
.001    0.001   .1      120.    .01     1.      0.
```

Figure 5-1. Comparison of PNNL's BREACH input file and a corrected one by the author.

6 Re-analysis of MCR Breach Flood

This section presents the procedure, result, and findings of the MCR breach flood re-analysis which was performed to support the concerns raised by the author. The re-analysis is based on a comprehensive modeling approach. The author considered all applicable empirical breach equations and multiple breach models. As was done by the STP, the re-analysis used a deterministic dam breach analysis that does not consider the condition of the MCR levee or the frequency of the potential MCR levee failures. The author postulated multiple breach scenarios in a conservative manner. The errors found in STP's and PNNL's BREACH modeling were fixed here. This study also considers the effects of sand core, cement blocks, and scour hole on the breach progression and the resulting flood.

Because there is no model that can handle both breach and flood simulation simultaneously, the author adopted a multi-step approach in series. The procedure used in the re-analysis is summarized as following:

(1) This study includes an estimation of breach parameters using all applicable empirical equations in a comprehensive manner. The breach parameters include width and time of breach as well as peak breach outflows.

(2) The author used two physically-based breach models, namely NWS BREACH (or just BREACH hereafter) and FLDWAV, to obtain and validate the breach parameters estimated by empirical equations and to generate breach outflow hydrographs.

(3) The author used the FLO-2D model to simulate the overland flow at the proposed plant site using the FLO-2D hydrodynamic model.

Because the RMA2 model which was used by STP that is not available to the NRC staff, the author used the FLO-2D model which is another numerical 2-dimentional hydrodynamic flow routing software. The following figure compares the different modeling approaches used for the MCR breach flood analysis. The following subsections describe details of each step in the re-analysis.

<u>STP's Approach</u>
Empirical Equ's → Breach Width and Time → FLDWAV → Hydrograph → RMA2
Use BREACH to validate width and time

<u>PNNL's Approach</u>
BREACH --→ Hydrograph → RMA2

<u>Re-analysis Approach</u>
Empirical Equ's → Breach Width and Time → FLDWAV → Hydrograph → FLO-2D
BREACH--- → Hydrograph → FLO-2D

Figure 6-1. Comparison of MCR breach analyses by STP, PNNL, and the author.

6.1 Concerns on the MCR Breach Analysis

In general, dam breach parameters are very sensitive on the resulting flood levels. This is true for the MCR breach case as shown by both STP's and PNNL's studies. At the beginning of the STP COL license application, STP postulated a very conservative breach scenario that results in a conservative design basis flood (DBF) level. This initial breach scenario is even more conservative than that for Units 1&2. However, STP changed the original scenario by adopting a narrow breach width through the FSAR Revision 2. The table below summarizes the different breach scenarios.

Table 6.1-1. Comparison of different breach scenarios and resulting design basis flood (DBF) levels.

Source	Distance from levee to the site (ft)	Initial Reservoir Water Level (ft MSL)	Breach Width (ft)	DBF Level at the site (ft MSL)
UFSAR for Units 1 & 2	600	50.9	2000	50.8
FSAR Rev. 0 for Units 3 & 4	2300	50.9	4757	47.6
FSAR Rev. 2 for Units 3 & 4	2300	50.9	417	40
Victoria ESP	1000	93.9 ft NAVD88	2034	90.6 ft NAVD88 (~10 ft in depth)

Note: The estimated DBF levels for the STP FSAR Rev. 0 andVictoria ESP is based on the simulation of the Delft3D model, while that of STP Rev. 2 is by the RMA2 model.

The change of the STP's breach scenario triggered an attention to the NRC reviewers, resulting in numerous RAIs, site audits, and independent confirmatory analyses. However, the author believes that this issue has not been resolved in the SER in a satisfactory manner. It is the author's opinion that both STP's and PNNL's breach flood estimates are neither correct nor conservative. To help resolving this issue, the author performed a comprehensive re-analysis.

Specifically, it is my opinion that both STP's and PNNL's approaches are biased by using selective methods and by ignoring conservative equations or methods without proper justification. In general, it is recommended by many papers and federal guides that the engineers who practice dam breach analysis should consider a variety of methods as dam breach estimations contain a considerable uncertainty. For instance, Brunner (2007) in USACE provides the following recommendation:

> "In general, several methods should be used to predict a range of breach sizes and failure times. It is recommended that the user uses all empirical equations, such as the three recommended above: Froehlich (1995b), MacDonald and Langridge- Monopolis (1984), and Von Thun and Gillette (1990). Additional regression equations can also be used, as well as any of the physically based computer models (the BREACH model is currently recommended). This will lead to a range of values for the breach size and

> failure times. A sensitivity analysis of breach parameters and times should be performed by running all of the parameter estimates."

Because the breaching process is complex and uncertain, the HEC-RAS manual (2008) suggests that the modeler should try to come up with several estimates of breach parameters and breach sizes, and then put together a matrix of potential breach sizes and times. One example would be use of two different sets of regression equations and one of the breach simulation models to estimate the breach parameters. Hydrologic Engineering Center in Davis California performed several dam breach studies using both the Froelich (1995) and the MLM regression equations, as well as the BREACH model (Fread, 1988). They noticed that all three methods give different answers for the breach dimensions and time. They ran each of these estimates as a separate trial within HEC-RAS to make reliable breach flood analysis.

Therefore, the author tried to use all available empirical breach equations and models, from which conservative breach scenarios are postulated in compliance with the GDC 2 requirement.

6.2 Assumptions Used in Postulating MCR Breach Scenarios

The author used the following assumptions to postulate MCR breach scenarios:

(1) The potential causes of the MCR breach are one or combination of the following three components:

- a piping failure caused by either seepage through the levee or its foundation,
- liquefaction of the foundation soils, or
- levee erosion from rain, flood, or others.

(2) The initial MCR water bevel is 50.8 ft MSL. The initial tailwater condition is dry – There is no rain or flood before the breach. STP determined the initial headwater level by considering the normal maximum MCR operating level of 49 ft MSL plus a margin for wind setup in the reservoir.

(3) A postulated MCR levee breach occurs at the nearest MCR levee section from the units 3 or 4 to maximize the DBF level. The exact placement of the breach section in this study was determined based on the FLO-2D model grid.

(4) Other than the Manning's roughness coefficient (n-value) used in the BREACH model, the conditions and parameter values of the embankment materials are the same as those used by STP.

(5) The author used the STP's BREACH model setup, but levee crest width and tailwater cross-section were corrected to match the field condition.

(6) The breach scenarios include the effects of scour hole, sand chimney and blanket, and cement blocks on the breach process.

By definition, dam is a wall built across a river or stream to stop water from flowing, whereas levee (or dike) is a wall used to manage or prevent water flow into specific land regions. Dams are commonly built on a rigid bottom and side foundation for the stability reasons, however most levees are build on a bare soils. Therefore, the mode of breach and parameters for levees (especially for long levees) could be different from those for dams as:

- Levee breach tends to be wider than dam breach, if the stored water in the reservoir is sufficient in quantity and head
- Levee breach accompanies a deep scour hole
- Tailwater in the breach downstream is characterized by a two-dimensional overland flow whereas dam breach flood is a steep channel flow.

The above factors were considered in this study. As the breach bottom roughness coefficient and scoring hole are critical to determining the beach process and resulting outflows, the following subsections focus on the discussion of the maximum levee breach width and score hole created during the levee breach.

6.2.1 Potential Maximum Levee Breach Width

Some hydrologists assume that there is a physically-acceptable maximum breach width regardless of reservoir-levee conditions. For instance, Von Thun and Gillette (1990) states in their unpublished paper that "the small database of large-dam failures tends to indicate 500 ft as a possible upper bound for breach width." However, this assumption is not true if we consider actual dam failure records. There are numerous reported cases where breach widths for dams and levees exceed far beyond 500 ft. The levee breach of the Martin Cooling Pond in Florida in 1979 is an example where the recorded breach width ended up to 600 ft. STP has been arguing that the MCR breach width of 417 ft is credible considering the historical data, such as the Teton dam failure case. Therefore, the author reviewed the historical breach records to determine the reasonable MCR levee breach width.

Breach Formation Factor

There are many factors that govern the breach process. MacDonald and Langridge- Monopolis (1984) found that the product of the breach outflow volume and the difference in elevation of the peak and base reservoir levels (e.g., V_w x h_w) is a reasonable variable for predicting breach eroded volume and breach width. They called this variable (V_w x h_w) as the breach formation factor. Their finding indicates that the breach process will be continued if there are sufficient water storage and head, or if there is no external factors that interrupt the breach process (e.g., hard rock bottom or abutment, structures, etc). Dams that have very large volumes of water and have long dam crest lengths will continue to erode for a long duration (Gee, 2008). That is, as long as a significant amount of water is flowing through the breach, there will be wider breach width and longer time. This continued breach are also found from the BREACH and FLDWAV simulations (It will be shown later).

Historical Maximum Breach Widths

The author reviewed the USBR dam breach database (Wahl, 1998) to see the maximum breach width that occurred historically. This database has over 108 historical dam breach cases from the world with 410 breach records - There are multiple breach estimation records for a breach in many cases. From this database, the recorded maximum dam breach width of 5800 ft was observed from the Machhu II dam failure in India. Excluding the Machhu II case, the maximum dam breach width is 820 ft, which is followed by 738 ft, 610ft, 551 ft, and so on. These data indicate that the recorded dam breach widths are wider than the STP's postulated MCR breach width of 417 ft.

There are many dam breach database available, however these databases do not include levee breach data or there is no known levee breach database. In the meantime, some reports or papers provide limited levee breach data for selective levee system(s) or region(s). For instance, Nagy (2006) provides 39 levee failure cases in the Eastern Europe Rivers, where

8000 ft is the maximum levee breach width. This maximum is followed by breach width of 1300 ft for the 2nd, 1000 ft for 3rd, and so on.

The URS (2007) report provided 14 historical levee breach cases from the California Delta Levee system, where 1018 ft is the maximum. This maximum is followed by breach width of 950 ft for 2nd, 926 ft for 3rd, and so on.

From these levee breach records, we can conclude that levee breach widths are generally wider than those of dams and that potential levee breach widths could be wider than 1000 ft. This conclusion supports the use of the MLM breach width equation for the MCR breach scenario.

6.2.2 Breach Scour Hole

Scour hole is not a common phenomenon in dam breaches because most dams are constructed on a hard rock foundation. That is why there are not many references that address the scour hole issue in dam breach. However, the scour hole is very common and an important issue in levee breach. The scour hole could increases the capacity (area) of breach outflows significantly and thus for flood levels. Some references address this specific issue (SFWMD, 1980; Zhang and Wang, 2001; Nagy, 2006; etc.).

Scour hole could be occurred right below the levee (Figure 6.2-1) and/or it could be happened at the headwater or tailwater zones or both (SFWMD, 1980; Figure 6.2-2 below). Nagy (2006) pointed out that developing a scour hole (pit) depends on subsurface (foundation) conditions as well as the mechanism of levee destruction. In terms of subsurface condition, scour holes are highly likely if the subsoil below levee is composed of grainy and transitional soils. The width of scour hole is usually smaller than the breach width if foundation is more compacted or less erodible than the levee itself.

SFWMD (1980) reported that the levee breach of the Martine Cooling Pond in Florida in October 1979 created a big scour hole having the dimension of:

- width of 450 ft
- length of 700 ft
- depth of 29 ft from the land surface (or the bottom of the levee), and
- Final levee breach width of 600 ft.

The scour hole for the Martine Cooling Pont has a U shape across the flow direction, having a wide top width but narrow bottom width (about 190 ft wide and 400 ft long). The maximum depth was appeared at slight downstream from the levee.

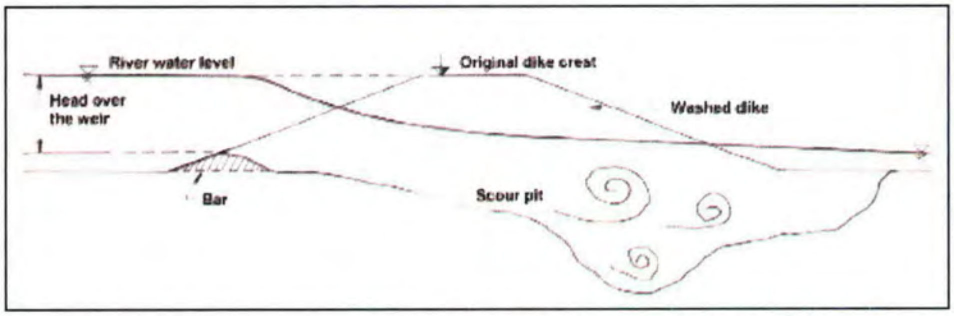


Figure 6.2-1. Typical cross section of a levee failure with scour hole (pit) (after Nagy, 2006).

Figure 6.2-2. Potograph of Elba River bank breach in 1976 (Nagy, 2006)

Zhang and Wang (2001) simulated the depths of scour hole from levee breach using a numerical model. The model is based on a solution of 2-dimentional shallow water and suspended sediment equations. They estimated that, under the breach width of 328 ft and the median grain side of 0.3 mm, the maximum scour hole depth from the surface is estimated to be about 20 feet. Their study indicates that the maximum scour hole occurs at the tailwater portion (e.g., after the toe) of levee or about 100 to 165 meters downstream from the invert of breach. They found that the hole occurs at the early stage of breach (1.7~10 minutes).

Based on the result of Zhang and Wang (2001), STP postulated a scour hole from the MCR breach at the toe of levee. This hole has width of 380 ft, length of 203 ft, depth of 20 ft from the surface elevation 29 ft MSL. They used the hole in estimating the volume of eroded materials to evaluate the effects of sedimentation deposition at the site on the DBF flood level. However, STP assumed no scour hole underneath the MCR levee thus no impacts of the hole on the MCR breach process and outflows. They justified that the road right after the levee prevent scour hole below the levee without proper justificaiton. STP's ignoring the scour hole in the MCR breach results in significant underestmation of the DBF level as my analysis shown later. This is a significant drawback in terms of a site safety analysis.

The author considered the following three cases to consider the scour hole effects in MCR breach:

- Case 1: No scour hole under the levee as in the case of the STP scenario
- Case 2: Extend the scour hole to the levee below, with an average hole depth of 10 ft (i.e., 0 ft on the invert and 20 ft at the toe of levee)
- Case 3: Extend the scour hole to the levee below, with an average hole depth of 20 ft (i.e., an average of 20 ft depths at both invert and toe of levee)

The pictorial descriptions of these cases are shown in the following figure. This study considers the above three cases in the breach flood analysis later.

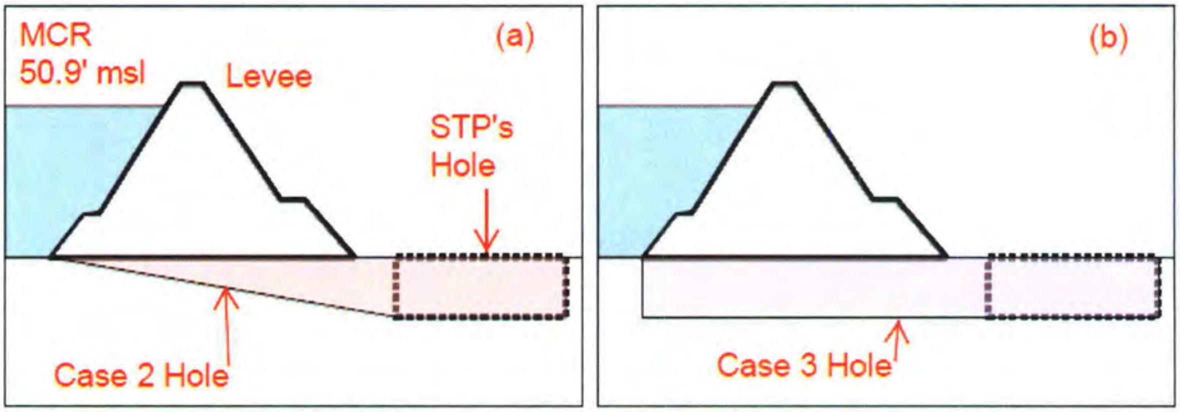


Figure 6.2-3. Schematic plots of MCR breach scenarios with different scour holes: (a) case 2 with 10 ft hole depth, (b) case 3 with 20 ft hole depth (not to scale).

6.3 Breach Parameter Estimation Using Empirical Equations

6.3.1 Dam Breach Analysis

In general, dam breach analysis includes dam breach simulation and downstream flood routing. The analysis uses either empirical regression equations or numerical models, or both. Because there is no model that can perform the two simulations together, hydrologists use a multi-step approach. For instance, breach parameters are estimated using empirical equations and them provided the parameters as input to breach outflow and flood routing models. The breach parameters include width (B) and time (Tf) of breach, rate (Qp) and time (Tp) of peak outflow, and breach outflow hydrograph (Q(t)) (refer to Figure 6.3-1). Several methods are available for estimating breach parameters (Wahl (1998):

- Simple regression equations to predict breach width and time
- Simple regression equations to predict peak outflows
- Physically-based breach simulation models, such as BREACH, FLDWAV, HEC-RAS, FLO-2D, etc.

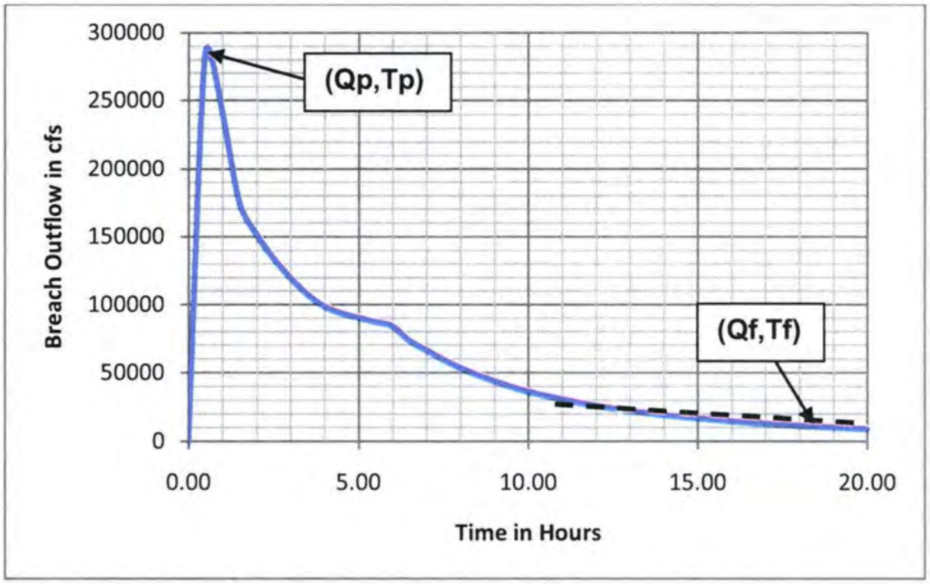


Figure 6.3-1 An example of breach outflow hydrograph simulated by the FLDWAV model for the breach width of 500 ft and the time to peak of 0.5 hours.

Regression equations estimates breach parameters as functions of one of more dam and reservoir parameters, such as storage volume (V_w), depth of water of depth of breach (h_w), etc. The equations are based on regression analyses of data collected from actual dam failures. Several federal agencies, such as USBR, provide the database of historical dam failures.

However, these databases are relatively lacking in data from failure of large dams, with about 75 percent of the case having a dam height less than 50 ft (Wahl, 1998). Further, they do not include levee failure records.

Chauhan et al. (2004) concluded, based on a simulation of DAMBRK (Fred,1984) and an empirical dam breach equation, that the use of breach parameters estimated from empirical equations as inputs to physical breach models such as DAMBRK typically results in significant overestimation of peak breach flows. However, their case study uses very limited breach conditions – It could end up different conclusion if one use different equations, models, and breach conditions. In fact, many dam breach simulation studies (e.g., Gee, 2008; Gee, 2010; Wahl, 1997, etc.) show that physical models give either higher or lower peak flows depending on site conditions.

As my study demonstrated later, physical models, like BREACH or FLDWAV, tend to produce overestimation of peak flow and time. It is my opinion that the breach parameter estimates using empirical equations are more credible than those by physical model because (1) empirical equations are based on real data, (2) validations of physical models are usually done with limited number of historical records or no recorded at all for future breach prediction, (3) breach parameter estimates using physical models are widely vary depending on model inputs which are by and large uncertain.

Breach side slope along the breach width and depth is a factor that specifies the shape of breach opening. However accurate predicting the breach side slope angle is of secondary importance to predicting the width and depth (Wahl, 1998). Therefore, this factor is not considered in the MCR breach analysis. As the side slope is insensitive to the breach outflow, this study used a fixed breach side slope angle of 45° (1h:1v slope) in BREACH modeling.

Most empirical breach time equations are for the final breach time (Tf). However, the breach time that is critical in flood hazard analysis is not Tf but the time to peak flow (Tp). Breach time estimated by physical breach models (e.g., BREACH, FLDWAVE, etc.) tends to be longer than those using empirical breach time equations. This long breach time may result in less conservative breach flood hazard.

Physically-based breach models are available to aid in the prediction of breach parameters. There are many reported physical breach models (Wahl, et al., 2008; Gee, 2008, etc.), including NWS-BREACH, SIMBA, HR-BREACH, FIREBIRD-BREACH, etc. Although none are widely used, the most notable is the National Weather Service BREACH models (Fread, 1988: NWS-BREACH or BREACH hereafter). Physical models are intended to simulate the hydraulic and erosion processes associated with flow from overtopping or piping failures of a dam and obtain breach parameters and breach outflow hydrographs.

Primary weakness of these physical breach models is the fact that they do not adequately model the erosion process that dominates the breaching of cohesive-soil embankments (e.g., Hahn et al. 2000). This is why many hydrologists recommend using multiple methods (both

empirical equations and physical models) in a dam breach analysis. However, both STP and PNNL used only the selective empirical equations or modeling. Instead, the author tried to use all available empirical equations and multiple models in a comprehensive manner.

6.3.2 Breach Parameter Estimation Using Empirical Equations

The listed below summarizes the empirical regression equations for estimating width and time of breach and peak breach outflow (Wahl, 1997):

- U.S. Bureau of Reclamation (1988)
  - Average breach width (m), $B_{avg} = 3\ h_w$
  - Failure time (hr), $Tf = 0.011\ B_{avg}$
  - Peak flow (m^3/s), $Qp = 19.1\ h_w^{1.85}$
- MacDonald and Langridge-Monopolis (1984)
  - Volume of eroded material (m^3), $B_{avg} = V_{er}/A = 0.0261\ (V_w\ h_w)^{0.769}/A$
  - Failure time (hr), $Tf = 0.0179 V_{er}^{0.364}$ for bounding, or $Tf = 0.0704\ (V_{er})^{0.2874}$ for best fit
  - Peak flow (m^3/s), $Qp = 3.85\ (V_w\ h_w)^{0.411}$ for bounding
- Von Thun and Gillette (1990)
  - Average Breach width, $B_{avg} = 2.5h_w + C_b$
  - Failure time (hr), $Tf = B_{avg}/(4h_w+61)$ for highly erodible dam materials
  - Peak flow (m^3/s), not available
- Froehlich (1995a, b)
  - Average breach width (m), $B_{avg} = 0.1803\ K_o V_w^{0.32} h_b^{0.19}$
  - Failure time (hr), $Tf = 0.00254\ V_w^{0.53}\ h_b^{-0.9}$
  - Peak flow (m^3/s), $Qp = 0.607\ V_w^{0.295}\ h_w^{1.24}$

where

h_w = depth of water above breach invert (m)
V_w = volume of water stored (m^3)
C_b = offset factor which is a function of reservoir size:
K_o = overtopping multiplier (1.4 for overtopping, 1.0 for piping)
h_b = height of breach (m), and
A = breach cross section area (m^2).

The MLM(1984) paper presents a bound failure time equation but not a best fit failure time equation. Therefore, the author obtained the above best-fitted breach time equation using 14 earth-fill dam failure data in the MLM's paper. The figure below shows the regression analysis where the historical data of erosion volume and time were log-transformed to fit a linear line as in the case of MLM's breach erosion volume equation. The linear regression equation was back-transformed to get an exponential breach time equation as shown above. The MLM best-fitted breach time equation is used in this study for consistency with other equations.

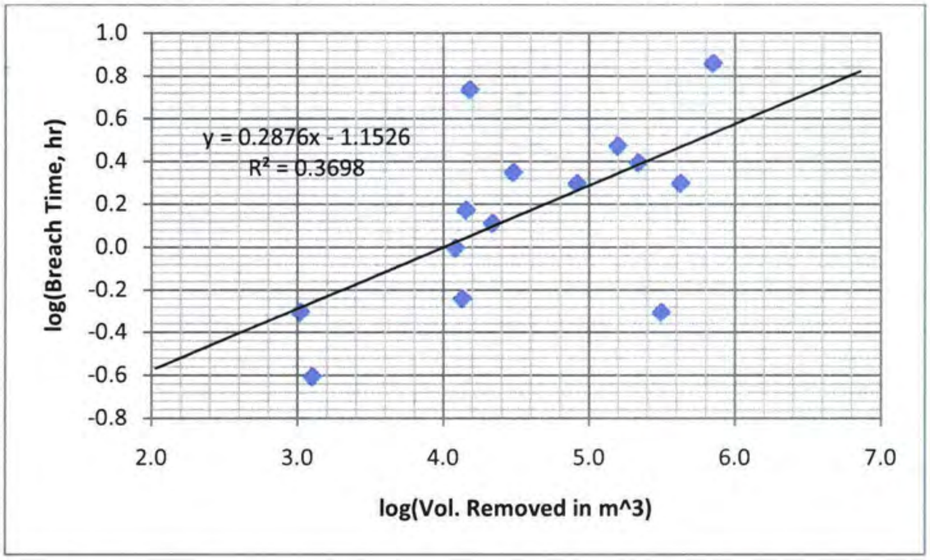


Figure 6.3-2. The best-fitted MLM breach time regression line that was obtained with the dam breach data provided by the MLM paper (1984).

The author used an initial MCR level before breach of 50.9 ft MSL and the bottom levee elevation of 29 ft MSL. The corresponding reservoir head (h_w) and storage volume (V_w) are 21.9 ft and 6.653x10^9 ft^3. Table 6.3-1 lists estimated breach parameters using the empirical equations above. The MLM breach width estimate in Table 6.3-1 is based on the scenario of no scour hole. The Table 6.3-2 compares the MLM breach widths for different scour hole depths. Table 6.6-3 lists peak breach outflow rates using 10 different empirical equations which are presented in Wahl's paper (1997). Some of the peak flow equations in Wahl (1997) were excluded in Table 6.3-2 because they are a bounding equation. However the envelop USBR peak flow equation is included because it is used in Table 6.3-1.

Table 6.3-1. MCR breach parameters estimated by four sets of empirical equations.

Equation	Breach Width (ft)	Failure Time (hr)	Peak Flow (cfs)
Bureau of Reclamation	66	0.66	22,607
MacDonald and Langridge-M	1736	2.54	217,000
Von Thun and Gillette	235	2.68	n.a. (1)
Froehlich	417	7.98	60,078

Note: (1) n.a. = not available.

(2) Cross section area of 5281 ft^2 at the breach bottom level of 29 ft MSL was used for the MLM breach width equation here.

Table 6.3-2. MLM breach width estimates using different scour hole scenarios

Scenario	Scour Hole Depth (ft)	Levee Erosion Volume (ft3)	MLLM Breach Width (ft)
Case 1	0 (no scour hole)	5281	1736
Case 2	10 ft	8754	1047
Case 3	20 ft	12309	745

Table 6.3-3. Breach preak flow equations and their estimated values for MCR.

Author	Equation	Qp(cfs)
Kirkpatrick (1977)	$Qp=1.268(h_w+0.3)^{2.5}$	5,754
SCS (1981)	$Qp=16.6(h_w)1^{.85}$	19,648
USBR (1982)-Envelop	$Qp=19.1h_w1^{0.85}$	22,607
Singh & Snorrason-1 (1984)	$Qp=13.4h_d^{1.89}$	46,105
Singh & Snorrason-2 (1984)	$Qp=1.776\ S^{0.47}$	458,501
Hagen (1982)	$Qp=0.54(S.h_d)^{0.5}$	826,102
MLM (1984)	$Qp=1.154(V_w{*}h_w)^{0.412}$	**217,247**
Costa (1985)	$Qp=0.981{*}(s{*}h_d)^{0.42}$	271,919
Evans (1986)	$Qp=0.72{*}V_w^{0.53}$	578,760
Freohlich (1997)	$Qp=0.607{*}V_w^{\ 0.295}\ h_w^{1.24}$	60,078
Average of 10 estimates		**251.000**
Average of last 5 estimates		**336,000**

Where, Qp = peak breach outflow in (m^3/s)
S = Reservoir storage (m^3), and
h_d = height of dam (m).

Based on the result of the above estimations, the following discussions were made:

- Both the Bureau of Reclamation and Von Thun and Gillette methods tend to underestimate breach width and peak flow rate because they are estimated as a function of storage or head only.
- USBR's dam breach manual (Wahl, 1998) states that the Froehlich's equation appears to be the best predictor for the breach widths less than 164 ft because the breach width records used to developed for the Froehlich equation is small. In other words, the Froehlich's breach width equation is not applicable to the MCR breach case.
- Therefore, this study considers the MLM method only. The MLML breach width range from 745 ft to 1736 ft with and without scour hole. The MLM estimates are comparable to the results of BREACH and FLDWAV simulation as shown later.

- Also the MLM peak flow estimate of 217,000 cfs is roughly consistent with an average (251,000) of 10 peak flow estimates shown below. It should be noted that the first five peak flows are underestimated because they ignore the MCR storage volume. Excluding these five, an average of peak flow is 336, 000 cfs.

The above discussions lead to a conclusion that the breach parameters estimated by STP (B=417 ft, Tf=1.7 hrs, Qp=130,000 cfs) are underestimated, resulting in an underestimation of the DBF level. This conclusion will be further validated using the simulation of numerical breach and flow models later. However, the STP' and PNNL's dam breach analyses ignored or omitted the above empirical equations-derived information partly or entirely without proper justification.

6.4 Breach Parameter Simulation Using the BREACH Model

The author used the NWS-BREACH model to supplement the breach parameter estimates by empirical equations. The BREACH model is probably the best known physically-based numerical breach model. BREACH model predicts breach parameters (size, shape, time of formulation) and breach outflow hydrograph emanating from a breached earth dam (Fread, 1993 & 2000). This model simulates coupling of the specific erosion processes and the flow process by breach. The BREACH model also accounts for the dynamic effects of the flow within the upstream reservoir using a simple mass-balance type reservoir routing and the tailwater effects on the flow through the breach. Because the BREACH model use only a limited immediate reach of the tailwater channel, additional flow routing model is needed for simulating the flood hazard far downstream.

The BREACH model uses the the Meyer-Peter & Muller (MPM) sedimentation equation for dam breach process and the 1-D Manning's uniform flow equation for the breach outflow. These equations are function of the Manning's roughness coefficient which has been known as one of the most critical parameters in the BREACH model (refer to Figure 6.4-1). STP uses n-value of 0.005 in their BREACH modeling, which seems too small for sediment-oriented breach flows. This n-value of 0.05 results in the peak breach outflow of 83,000 cfs which is also small compared to the estimates by other methods.

PNNL (2011) selected n-value of 0,075 without justification. PNNL performed a sensitivity analysis, but part of their sensitivity analysis is erroneous due to a faulty BREACH setup. Therefore, the author investigated the proper n-value for the MCR breach, followed by a supplemental sensitivity analysis of the MCR BREACH model.

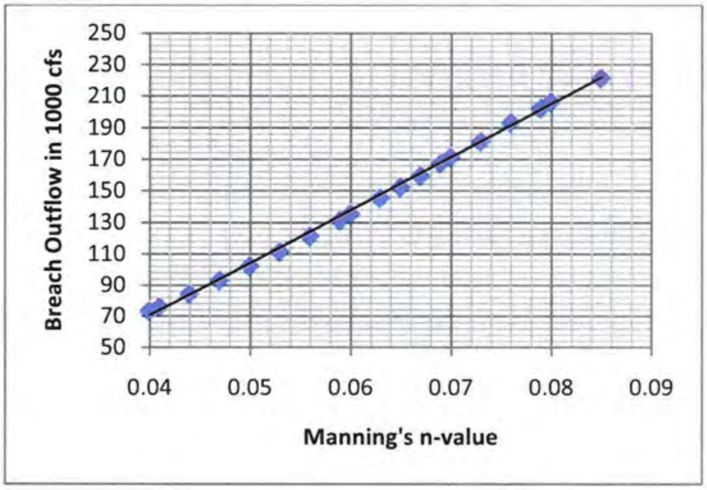


Figure 6.4-1. The relation between Manning's n-value and breach peak outflow obtained from BREACH simulations (without scour hole effects).

6.4.1 Manning's Roughness Coefficients

There are many referenced n-values applicable to channel and overland flows. However, there is no specific guidance that directs to select n-value for dam breach simulations. STP's FSAR states that, for BREACH modeling, an n-value within the range from approximately 0.025 to 0.05 was considered reasonable, while a higher n-value of 0.08 is still considered within the range of feasibility. The n-value of 0.075 is used in SER. The BREACH manual proposed to use the Straler's method to estimate n-value. However, the estimated n-value with this equation is too small because the size of the embankment material (silty clay) is small (0.001 mm).

Chow (1969) states that n-values for sediment laden flow is higher than without it. Since n-value in the BREACH model is critical in breach process, this subsection estimate the n-value for the MCR breach considering the embankment materials, sand chimney, and cement blocks. The author used the Chow's method to estimate the n-value for the MCR breach. The estimated value was compared with other literature values.

<u>Chow's Method to Estimate n-value</u>

This report estimated Manning's n-value using the method presented by Chow (1959; Arcement and Schneider, 1984). Table 5-5 in Chow (1959) provides a procedure for estimating specific n-value considering the effects of bed materials and shape of channel as:

$$n=(n_b+n_1+n_2+n_3+n_4)m \qquad \text{(Equ. 6.4-1)}$$

where

n_b is the base n value
n_1 is the irregularly factor
n_2 is the channel cross section factor
n_3 is the obstruction factor
n_4 is the vegetation factor, and
m is the meandering factor.

The parameters in equation (6.4-1) are estimated based on the above two references as following:

- n_b is 0.02 for earth bed materials which consisted of sand with the median diameter of 0.4 mm.
- n_1 is 0.015 for an average of moderate and severe irregular channels caused by badly eroded or sloughed sides of canals. The maximum n_1 for severe irregularity is 0.02.
- n_2 ranges from 0.01 to 0.015 for frequently alternating of channel cross section due to the constriction at the breach invert area and wide expansion at the breach outlet. The rapid contraction and expansion create highly turbulent breach flow.

- n_3 ranges from 0.02 to 0.03 for appreciable obstruction caused by the littering of clay cement blocks that fall and cover about 40% of the breach surface area during the breach process.
- n_4 is 0.005 for small vegetation. The maximum n_4 is 0.01.
- m is 1 for minor (minimum) meandering.

Using the above, the low and high n-values are computed as 0.07 and 0.085, respectively, with an average n-value of 0.0775. These estimated values are compared with other tabulated and referenced n-values as the table below.

Table 6.4-1. Comparison of the Manning's roughness coefficients (n-values) for difference sources.

Source	n-value		
	Minimum	Medium	Maximum
FSAR	0.025	0.05 (used)	0.08
SER	0.025	0.05	0.075 (used)
Chow Method (this study)	0.07	0.0775	0.085
Table 5-6 in Chow (1959) – major stream (B>100 ft) with irregular and rough section	0.035	-	0.10
FLO-2D MANUAL – overland flow	-	0.07	-
Estimated based on the MLM peak flow of 217,000 cfs	-	0.085	-

In summary, n-values for the breach process could range from 0.025 to 1.0. The estimated n-value in this study for the MCR breach considering the breach condition, sand chimney, and cement blocks is 0.0775, but the BREACH model in this report uses an n-value of 0.075 in line with the SER.

6.4.2 BREACH Sensitivity Analysis

The BREACH model uses the following input parameters:

- Dam materials: grain size, porosity, unit weight, n-value, frictional angle, cohesive strength, and D90/D30 ratio for both inner and outer materials

- Dam properties: (1) for clay materials, PI index, critical shear stress, and critical stress coefficient; (2) slope of upstream and downstream faces, and slope of core; (3) for grass cover, length, condition, max erosion velocity, max sediment concentration

- Breach parameters: ratio of initial width to depth, width of dam, tailwater slope, particle size (D50), ration of D90 and D30, max bottom width, and max top width

- Elevation: initial reservoir level, top and bottom elevations of dam, initial piping elevation, and spillway crest.

- Others: tailwater cross section, simulation time step, and output control.

PNNL performed a BREACH sensitivity analysis to determine the sensitivity of inputs on the MCR levee breach process and outflows (PNNL, 2011: refer to the PNNL's calpack). Based on the sensitivity analysis, PNNL concluded that:

- The BREACH model predictions are very sensitive to n-value.

- The BREACH model predictions are slightly sensitive to internal frictional angle.

- The breach predictions for peak flow and time and breach width are fairly insensitive to initial piping elevation, cohesive strength, frictional angle,

- The model predictions are very sensitive to n-value.

However, PNNL made erroneous sensitivity conclusions for (1) length of levee as not at all sensitive; and (2) tailwater cross section as slightly sensitive. Therefore, the author performed an additional sensitivity analysis for n-value, levee length, tailwater cross section, and scour hole as described below.

<u>Formulate a Base Run Scenario</u>

To start with, the author postulated the following four MCR breach scenarios for which the result of BREACH simulations is summarized in the table below:

- FSAR scenario: {N=0.05,C=300 lb/ft2, phi=20^{O}}

- SER scenario: {N=0.075,C=200 lb/ft2, phi=15^{O}}, but all other parameters are the same as those for the FSAR scenario.

- RUN1: A corrected SER scenario with tailwater cross section and max levee length of 3000 ft. Note: MCR BREACH gives a runtime error if width is greater than 3000 ft.

- RUN2: RUN 1 with a 10-ft scour hole.

Table 6.4-2. Comparison of BREACH simulations with different MCR breach scenarios..

Run	Condition	Peak				Final Breach[(2)]			
		Qp	Tp	Breach Width		Qf	Tf	Breach Width	
		kcfs	hour	Top, ft	Bot., ft	kcfs	hour	Top, ft	Bot.,ft
FSAR	n=0.05	83	6.25	412	361	26	22.51	498	446
SER	n=0.076	128	1.99	574	463	23	20.51	747	637
RUN1	Corrected	194	3.29	989	878	24	17.39	1259	1149
RUN2	Scour hole	269	2.12	703	562	26	21.64	861	720

Notes:

- Kcfs is the 1000 cubic feet per second.
- RUN 2 is the base run for the flowing sensitivity analysis.

The tables above and below provide peak breach parameters as well as those at the end of the breach process in order to compare those using empirical equations. The final breach time is almost infinite, but they were determined using the criteria of (1) the difference between the headwater level and tailwater level (e.g., HY-TWD parameter from BREACH output) become less than 1 ft or (2) breach outflow is less than 27,000 cfs which is about 10% of RUN 2 peak outflow. The peak parameter values for the "SER run" are from the PNNL calpack – These are slightly different from the simulation of this study. From this simulation, the following discussions are made:

- The result of this simulation indicates that the corrected BREACH model produces higher peak outflow, and even higher if the effects of scour hole are considered.

- The breach time simulated by the BREACH model tends to be longer than the ones by empirical equations. However, the tail of the outflow is not significant in determining the DBF level, thus ignored.

- The breach process continues after the peak flow, but the rate of the breach process is relatively slow compared to that before reaching the peak outflow.

<u>BREACH Sensitivity Runs</u>

Table 6.4-3. Sensitivity of Manning's n-value to BREACH model results.

Run ID	n-value	Peak				Final Breach			
		Qp	Tp	Breach Width		Qf	Tf	Breach Width	
		kcfs	hour	Top, ft	Bot., ft	Kcfs	hour	Top, ft	Bot.,ft
R11	0.10	323	1.06	820	679	60	9.68	1018	877
R12	0.08	280	1.52	723	583	46	12.24	892	751
BASE	0.075	269	2.12	703	562	26	21.64	861	720
R13	0.06	224	2.96	623	483	31	16.5	745	604
R14	0.04	139	6.20	481	340	32	19.4	557	416

Table 6.4-4. Sensitivity of scour hole size to BREACH model results.

Run ID	Hole Depth	Peak				Final Breach			
		Qp	Tp	Breach Width		Qf	Tf	Breach Width	
	Ft	kcfs	hour	Top, ft	Bot., ft	kcfs	hour	Top, ft	Bot.,ft
R21	0	195	3.63	1029	918	26	16.40	1676	1565
R22	5	252	3.04	875	749	26	18.93	1107	981
BASE	10	269	2.12	703	562	26	21.64	861	720
R23	15	271	1.80	595	439	26	24.65	716	560
R24	20	267	1.64	518	347	26	14.8	620	450

Notes: R21 and RUN1 in Table 6.4-2 are identical.

Table 6.4-5. Sensitivity of tailwater cross section to BREACH model results.

Run ID	Tailwater Width	Peak				Final Breach			
		Qp	Tp	Breach Width		Qf	Tf	Breach Width	
	Ft	kcfs	hour	Top, ft	Bot., ft	kcfs	hour	Top, ft	Bot.,ft
BASE	3000	269	2.12	703	562	26	21.64	861	720
R31	2000	244	0.77	611	470	24	24.42	849	709
R32	1500	219	0.78	570	430	24	24.92	798	658
R33	1200	204	0.98	570	429	25	33.39	781	641
R34	1000	195	0.79	526	386	26	25.14	771	630
R35	800	159	1.10	489	349	23	29.33	679	540
R36	600	122	0.80	388	248	25	29.84	567	426

Note: (1) On these sensitivity runs, levee length was fixed to 3000 ft.

(2) The tailwater section width greater than 3000 ft give a runtime error on the MCR BREACH simulation.

Table 6.4-6. Sensitivity of levee length to BREACH model results.

Run ID	Dyke Length	Peak				Final Breach			
		Qp	Tp	Breach Width		Qf	Tf	Breach Width	
	Ft	kcfs	hour	Top, ft	Bot., ft	kcfs	hour	Top, ft	Bot.,ft
BASE	3000	269	2.12	703	562	26	21.64	861	720
R461	600	265	0.62	600	488	26	15.39	600	488
R462	500	220	0.47	500	388	25	16.74	500	388
R47	400	167	0.39	400	288	25	22.04	400	288

Note: On these sensitivity runs, width of tailwater cross section was fixed to 3000 ft.

The above four tables summarize the result of sensitivity analyses for n-value, scour hole depth, width of the bottom tailwater section, and levee length, respectively. From this sensitivity runs, the following discussions are made:

- The BREACH model predictions are very sensitive to n-value. This was also noted in the PNNL's calpack and Figure 6.4-1.
- The BREACH model predictions are very sensitive to the scour hole depth. Ehe effective of scour hole is nonlinear - The peak outflow rates increase with increasing hole depth. The flow rates are peaked (271 kcfs) at the hole depth of 15 feet, then decrease thereafter. It seems that this peak flow is the probable maximum MCR breach outflow, if scour hole is considered in the breach process.
- Both peak time and breach width is inversely proportional to the hole depth. The deeper the scour hole is, the more the initial breach outflows is. However, scour hole reduces breach width. In conclusion, considering the effects of breach scour hole induce high peak breach outflows and more severe flood hazard than without considering it.

- In contrast to the PNNL's conclusions, the sensitivity analysis by this study demonstrates that BREACH predictions are very sensitive to both levee length and the width of tailwater cross section. These are not variable. However, use of correct and realistic values for these two inputs is needed in BREACH modeling.

6.5 Breach Outflow Simulation Using the FLDWAV Model

The FLDWAV model (Fread, 1993) is a generalized unsteady-flow model for open channels. It replaces the DAMBRK, combining its dam breach simulation capabilities and providing new hydraulic simulation procedures. FLDWAV can simulate dam failures caused by either overtopping or piping. The program can also represent the failure of two or more dams located sequentially on a river. The program is based on the complete equations for unsteady open-channel flow (St. Venant equations). This model has options to simulate various external and internal boundary conditions and hydrologic structures.

Fread (2000) states that "BREACH should be used judiciously and with caution - BREACH model is intended to be an auxiliary method for determining the breach parameters and should be used in conjunction with statistical and range of magnitude data from historical breaches." Therefore, the BREACH model has not been directly incorporated into FLDWAV. However, FLDWAV can easily simulate breach outflow hydrographs using breach parameters (width and time) as input. This feature gives an advantage of bypassing the burden of identifying the BREACH input parameters, especially the Manning's n-value. After setting up the FLDWAV model for the MCR breach case, the author performed a comprehensive sensitivity analysis to understand the behavior of the MCR FLDWAV model.

<u>FLDWAV Model Set-up</u>

The MCR FLDWAV model was set up with available data from FSAR and RAI responses. The MCR FLDWAV model consists with three hydrologic components: MCR reservoir routing, MCR levee breach, and tailwater routing. All of these hydrologic components are simulated in a 1-dimentional way because FLDWAV does not have capability to simulate 2-D flows. The MCR FLDWAV model uses five channel cross sections: The first cross section is for MCR reservoir with levee, and remaining four are for tailwater reaches.

First, MCR reservoir is used as an upstream boundary condition. Relevant reservoir input to FLDWAV include MCR stage-surface area rating curve, starting and bottom reservoir elevations, and upstream inflow (discharge) boundary condition. There is no actual upstream inflow into the MCR during the breach, but small inflow values were input to the model to provide a stability of the channel routing - FLDWAV gives a runtime error without this fictional input.

Second, levee breach input include elevations of top and bottom of levee, weir coefficient, breach time, breach width, initiating reservoir level, reservoir bottom level, levee crest length, initial piping level, and breach side slope (1h:1v). These values are same as those for the MCR BREACH model. Again, width and time of breach is the most critical parameters in determining breach outflows.

Third, tailwater flow condition was assumed as a 1-D flow on a wide channel (4 miles width) to mimic the 2-D overland flow condition at the power block area. This tailwater condition is more realistic than that of the MCR BREACH model that uses only one tailwater cross section with a

width of 3000 ft. Tailwater input include number of cross sections, Manning's n-value for channel (specified from 0.03 to ~0.08, but actual n-values are calculated by the model internally depending on the condition and depth of flows), length and slope of each channel reach, and cross section geometry.

After setting up the MCR FLDWAV model, it was tested for the STP's breach scenario (e.g., B=415 ft, Tf=1.7 hours). The test run provide the same peak outflow (128,000 cfs), validating that the model was set up correctly. The author performed a comprehensive sensitivity analysis in order to understand the behavior of the MCR FLDWAV model.

FLDWAVE Sensitivity Runs

The author simulated the MCR FLDWAV model with different widths and times of breach. The upper bound of the breach parameters are determined conservatively (5000 ft width and 3 hours breach time). For discrete values of breach parameters, 42 FLDWAV runs were performed. The following two tables list simulated peak outflows and times to peak. They were also plotted in the following two figures.

Tp\B_avg	500 ft	750 ft	1000 ft	1500 ft	2000 ft	3000 ft	5000 ft
0.5 hr	285	415	539	735	830	933	1077
1.0 hr	277	391	466	535	590	670	779
1.5 hr	265	331	369	425	469	534	635
2.0 hr	241	280	309	357	398	454	523
2.5 hr	210	244	271	312	348	397	475
3.0 hr	189	219	241	280	306	361	426

Table 6.5-1. Peak breach outflows (in 1000 cfs) simulated by the MCR FLDWAV model with different breach widths and times.

Tp\B_avg	500 ft	750 ft	1000 ft	1500 ft	2000 ft	3000 ft	5000 ft
0.5 hr	0.5	0.5	0.5	0.5	0.5	0.425	0.35
1.0 hr	1.0	1.0	1.0	0.9	0.8	0.7	0.6
1.5 hr	1.5	1.5	1.35	1.20	1.05	0.9	0.75
2.0 hr	2.0	1.8	1.6	1.4	1.3	1.1	0.9
2.5 hr	2.375	2.125	1.875	1.625	1.5	1.375	1.125
3.0 hr	2.7	2.4	2.1	1.95	1.65	1.5	1.2

Table 6.5-2. Peak breach time (in hour) simulated by the MCR FLDWAV model with different breach widths and times.

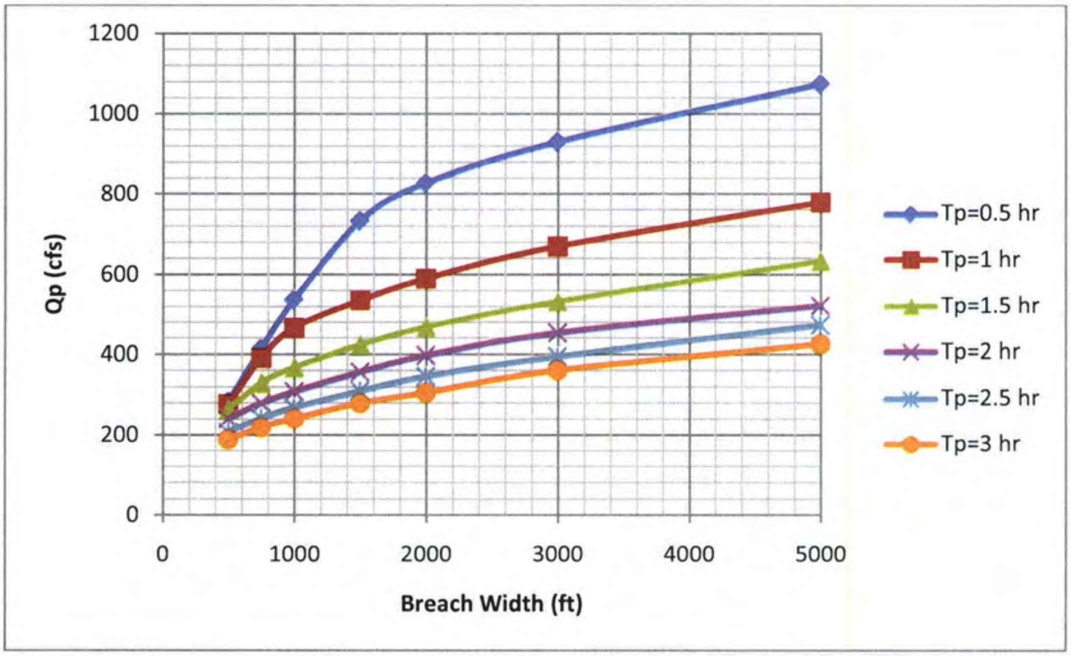


Figure 6.5-1. Breach peak outflows estimated by FLDWAV with different breach widths and times.

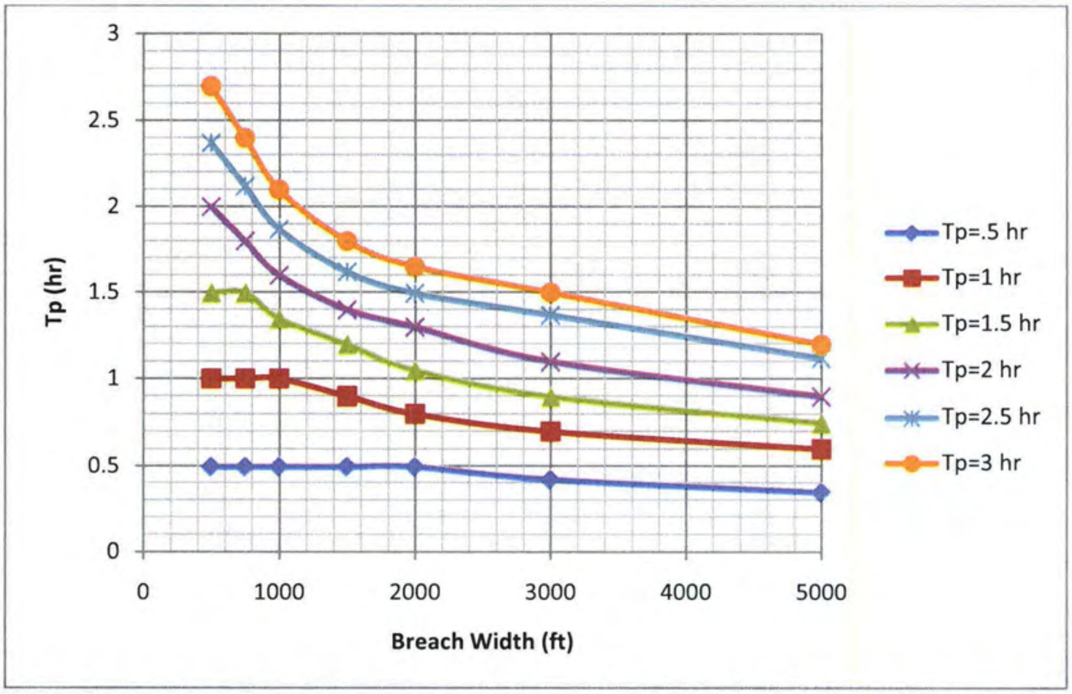


Figure 6.5-2. Breach peak outflows estimated by FLDWAV with different breach widths and times.

From these sensitivity runs, the following discussions were made:

- The peak outflow is proportional to breach width, but the rate of increase is nonlinear. The peak outflow increases shapely for the breach width less than 2000 ft, but the rate of increase is reduced thereafter.

- The pink color cells in Table 6.5-2 are the case where FLDWAVE-simulated breach peak time is less than the corresponding peak time input. The reduction is more significant for wider breach widths. The reduction indicates that peak time in FLDWAVE model input is unrealistically small so that the FLDWAVE model adjusts the outflow and time to peak based on the breach condition and geometry (breach width and others).

- The two figures above are useful in determining the peak outflow and time given the breach width and time obtained from empirical equation. In cases where peak outflow and time are estimated independently, it is necessary to filter and adjust the parameters using the FLDWAV runs. Figure 6.5-1 and 6.5-2 are useful in adjusting the peak outflow and time as shown in Subsection 6.6.6.

6.6 Flood Routing Using the FLO-2D Model

In general, dam breach flood analysis in a river valley or channel has been performed with 1-dimensional (1-D) flood routing models such as HEC-RAS or FLDWAV. In such cases, dam breach flood wave is confined laterally by natural abutments and well-defined downstream river channels, therefore an implicit assumption of velocity vectors normal to the river cross-sections is reasonable for dam breach analysis. However, levee breach flood routing for a large off-stream reservoir constructed on flat ground must be done with a 2-dimentional (2-D) simulation with velocities and flows dispersing in any direction. The MCR levee breach flood at the power block area should be handled by a 2-D hydrodynamic model. The main difference between 1-D and 2-D simulations is that, channel cross sections for 1-D simulation are replaced with a continuous topographic surface for 2-D simulation.

The MCR levee breach flood analysis in this study is based on a combined approach of 1-D simulation of breach outflows and 2-D routing of overland flow after the breach. Because the RMA2 model that was used by STP is not available to the NRC staff, the author used the FLO-2D model. FLO-2D is a dynamic routing model that simulates channel flow, unconfined overland flow and street flow. It can simulate a flood over complex topography and roughness while reporting on volume conservation which is the key to accurate flood routing. FLO-2D uses a full dynamic wave momentum equation and a central finite difference routing scheme with eight potential flow directions over a system of square grid elements. The equations of motion in the FLO-2D finite difference model grid are defined as a quasi two-dimensional model, which are solved by computing the average flow velocity across a grid boundary one direction at a time. FLO-2D does not distinguish between subcritical and supercritical flow because the momentum equation used in the model has no restrictions when computing the transition between the flow regimes. Because of this advantageous feature, the FLO-2D model has been widely used in simulating levee breach floods.

6.6.1 FLO-2D Model Set-up

The author set up the MCR FLO-2D model to simulate the MCR breach induced flood at the power block area. The FLO-2D model has options to simulate various types of flows, including overland flow, channel flow, flood plain, levee, dam and levee breach, hydraulic structures, street flow, rain-runoff-infiltration process, sediment transport, mud and debris flow, and others. However, the MCR FLO-2D model utilizes only the overland flow option and limited options for levee and levee breach because it uses the breach outflow hydrograph simulated by the FLDWAV model as upstream boundary inflow. All other options were omitted, making the MCR FLO-2D model very simple and effective in running. The input to the MCR FLO-2D model are as following:

- Static data: surface elevation and n-value in space, drainage features (location, width, depth), features of ponds and reservoirs (extend, elevations, levee information), features of structures and roads (location, elevation, etc).
- Boundary conditions: upstream boundary inflow hydrograph, location of outflow boundary nodes – they are not sensitive to the DBF level.
- Levee information: Location, top and bottom elevation

- Breach information: location (nodal points) and width of breach
- Numerical tolerance input: surface detention factor, allowable percent change in flow depth, and dynamic wave stability factor.

The boundary of the MCR FLO-2D model was determined by a trial and error so that the model area is small enough to run efficiently but is large enough to avoid the effects of the downstream boundary on the estimated DBF level (Figure 6.6-1). The model domain in this study is larger than that of the STP's RMA2 model. The model domain is divided into uniform, square grid nodes (200 ft resolution). It has a total of 29948 nodes. The nodal points for structures and MCR levee were assigned in the model.

Breach failure location was identified as the closed levee section from the proposed units 3 and 4. The model was set up so that the location of the breach section is used as an inflow boundary. This input boundary feature enable to exclude the entire MCR area, which reduce model run time significantly. The MCR FLO-2D model uses the breach outflow hydrograph simulated by the FLDWAV model as input as was done in the STP's RMA2 model.

At the site, the typical n-value of 0.045 was assigned. The author performed a sensitivity analysis of the MCR FLO-2D model with changing n-values from 0.025 to 0.08. The result shows that the n-value is not sensitive to the DBF levels – The change of DBF levels is only 0.36 ft which is substantially smaller than the change by breach parameters. This is because FLO-2D has a flow depth variable n-value function so that the n-values are adjusted by the model internally rather than using the input n-values.

Other model parameters for MCR, levee, and structures are the same as those used in the STP's RMA2 model. The run time of the MCR FLO-2D model is quite extensive – Simulation of a 20-hour MCR breach flood took more than 10 hours. Most of the cases, the MCR FLO-2D model simulations were stopped externally by the author after 5 hours in model time because the maximum flood levels at the site occurs 2 to 3 hours.

This study, like the STP's analysis, used a sunny day levee breach which is caused by seepage through levee or foundation, or slide of levee materials. That is, MCR antecedent rain or storm surge effects on the MCR breach flood were not considered in the MCR FLO-2D model. Evaporation and recharge losses of the MCR breach water were also ignored in flood routing as they are insignificant in determining the DBF level.

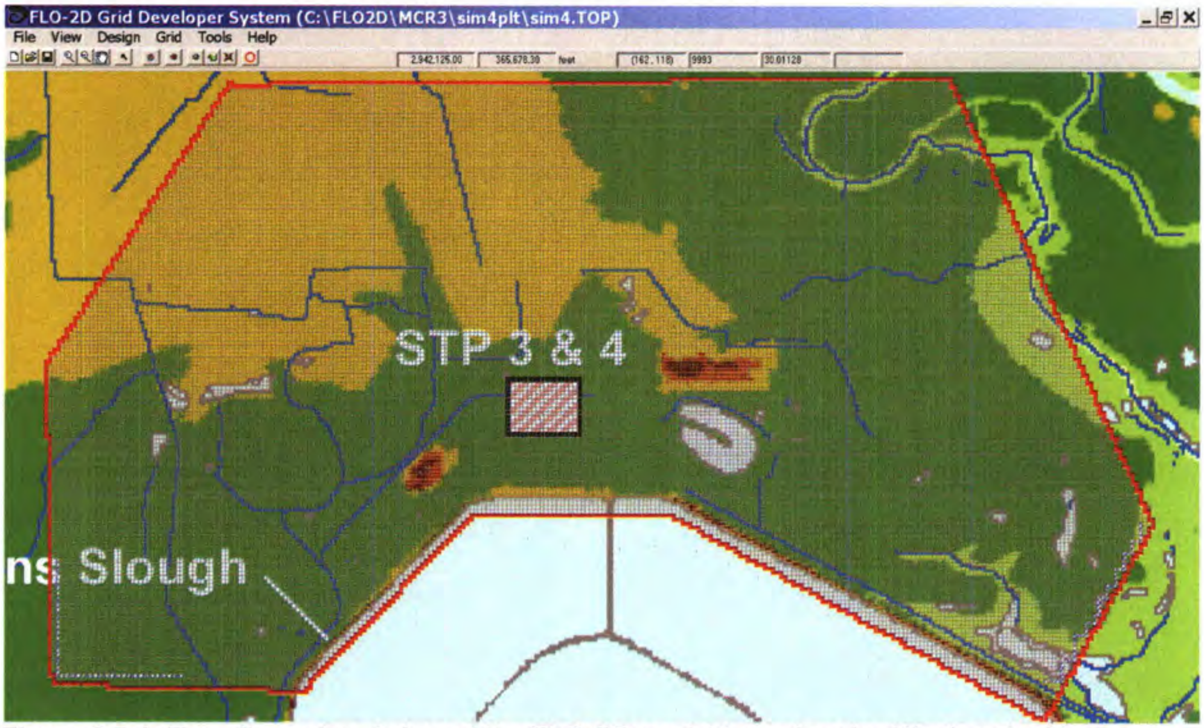


Figure 6.6-1. Two-dimensional view of the FLO-2D model boundary (red line) and grid.

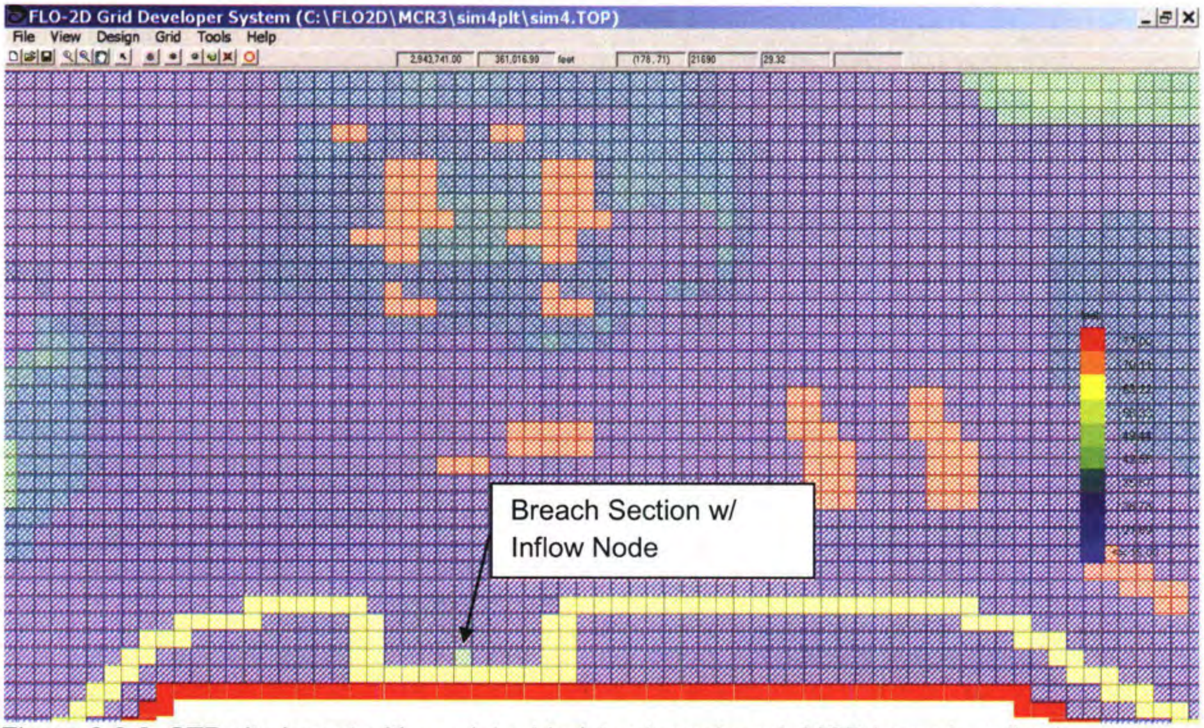


Figure 6.6-2. STP site layout with model grids (structures in red, MCR levee in yellow).

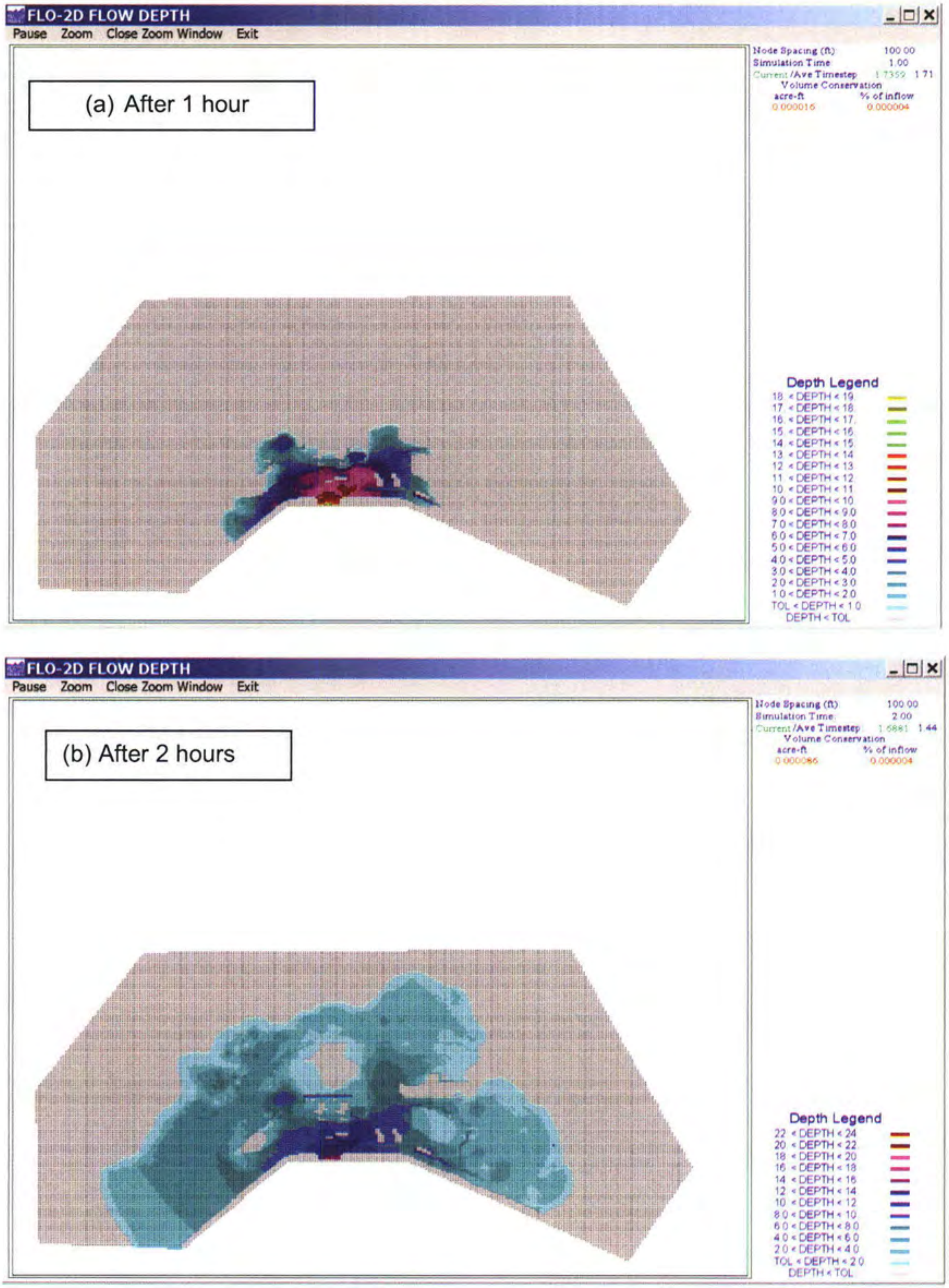


Figure 6.6-3. An example of breach flood propagations simulated by MCR FLO-2D at (a) 1 hours after initiating a breach (b) after 2 hours, and (c) after 3 hours.

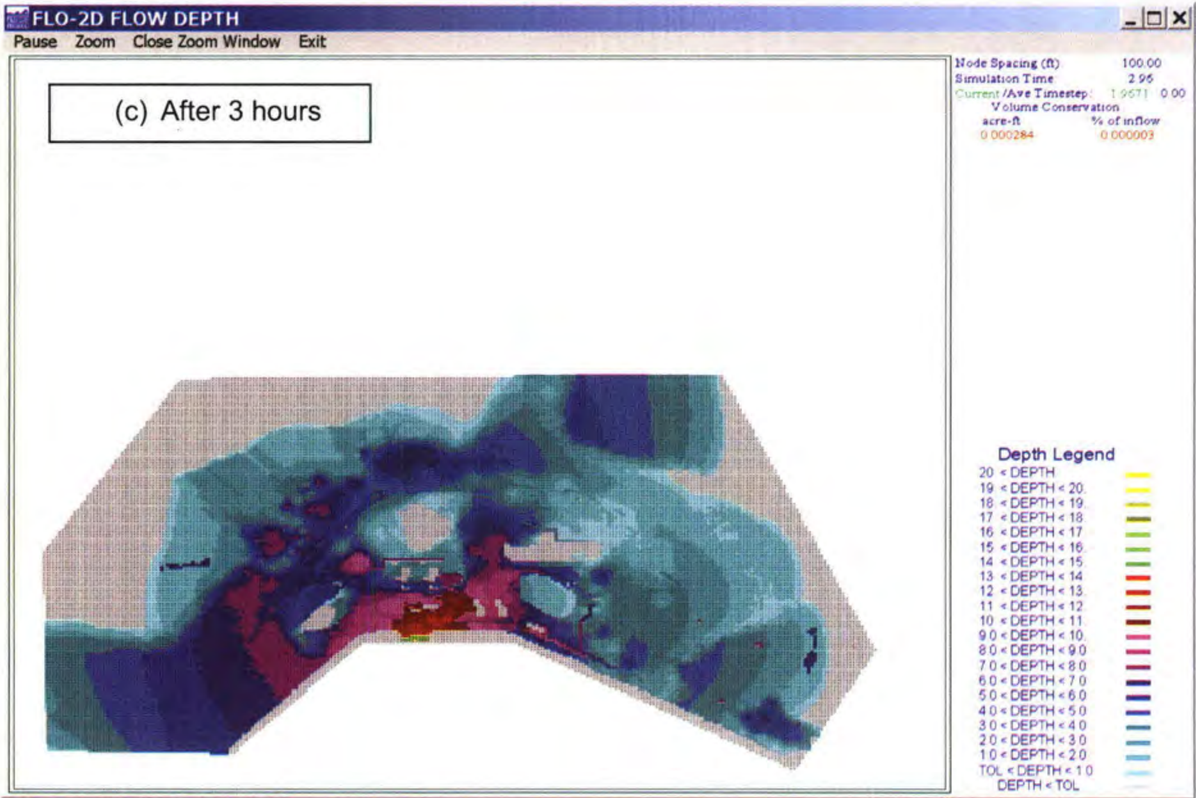


Figure 6.6-3 (continue)

This report includes some simulation maps to help understand the behavior of the MCR FLO-2D model. These maps are snapped from the FLO-2D pre/post processing software during the simulation.

Figure 6.6-3 shows a typical MCR breach flood propagation in space simulated by the MCR FLO-2D, where the propagation is displayed in terms of the maximum flow depths. This example is for the simulation of the STP's MCR breach scenario case where breach width is 417 ft and breach time is 1.7 hours. As shown from this figure, levee breach water flows towards the north with inundating the site, divides into the East and West directions almost equally, and then eventually flows to south onto either the Colorado River or the Little Robin Slough.

Figure 6.6-4 displays the land surface elevation map used in the MCR FLO-2D model. The land surface elevation data were provided by STP, which are digitized to the MCR FLO-2D model grid cells using the fLO-2D pre-processing software. Some of the cell elevations were adjusted based on the configuration of the drainage system, MCR reservoir-levee system, and proposed plant facilities to incorporate control of flow directions.

Figure 6.6-5 and 6.6-6 display simulated maximum flood elevation and maximum flood velocity maps, respectively, using the MCR FLO-2D model. The simulation condition of these maps is the same as that of Figure 6.6-3 (STP's breach scenario).

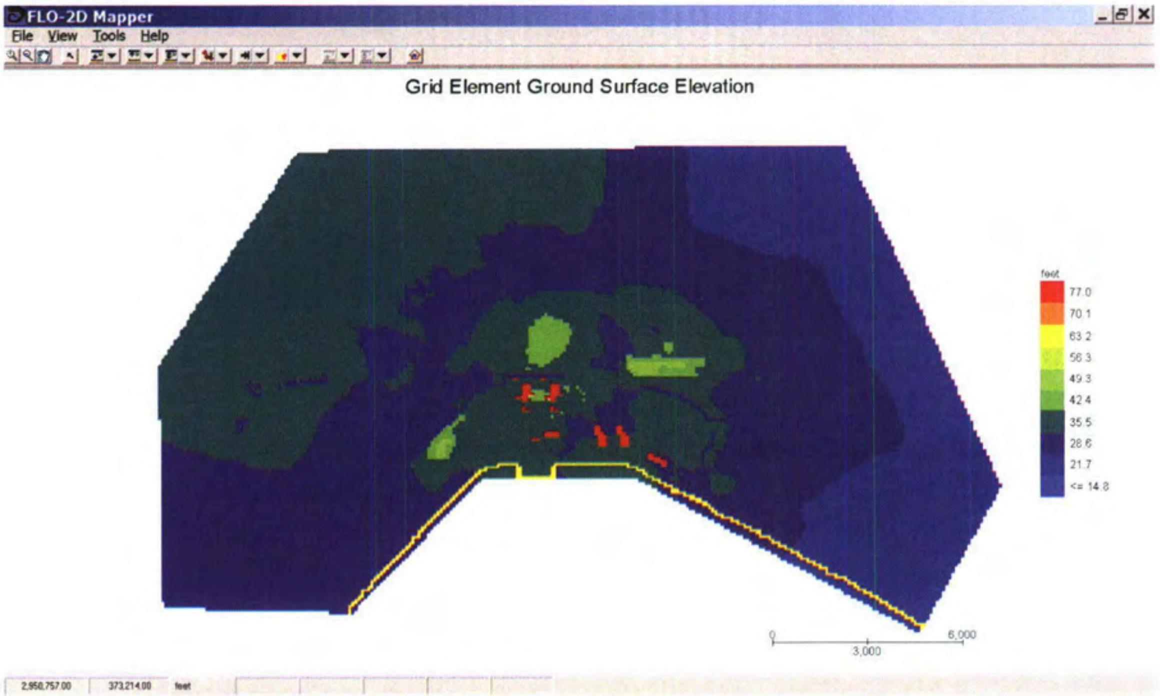


Figure 6.6-4. MCR FLO-2D surface elevation map.

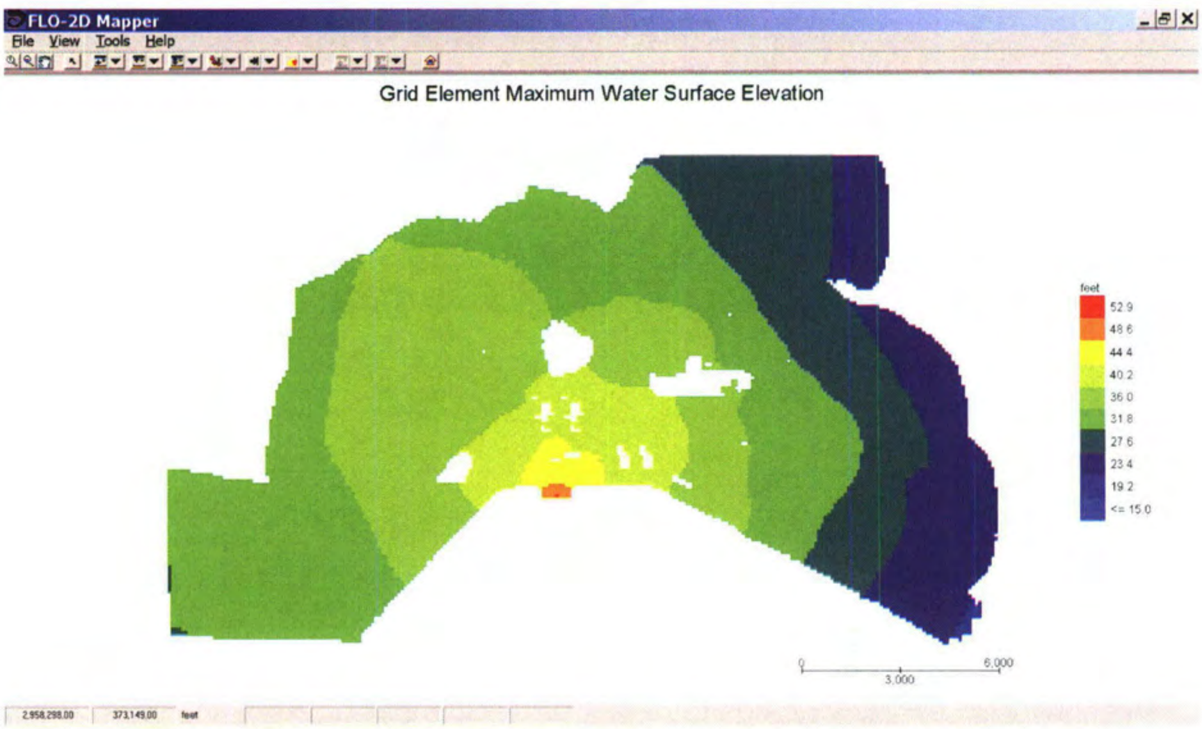


Figure 6.6-5.An example of the FLO-2D simulated maximum water surface elevation map.

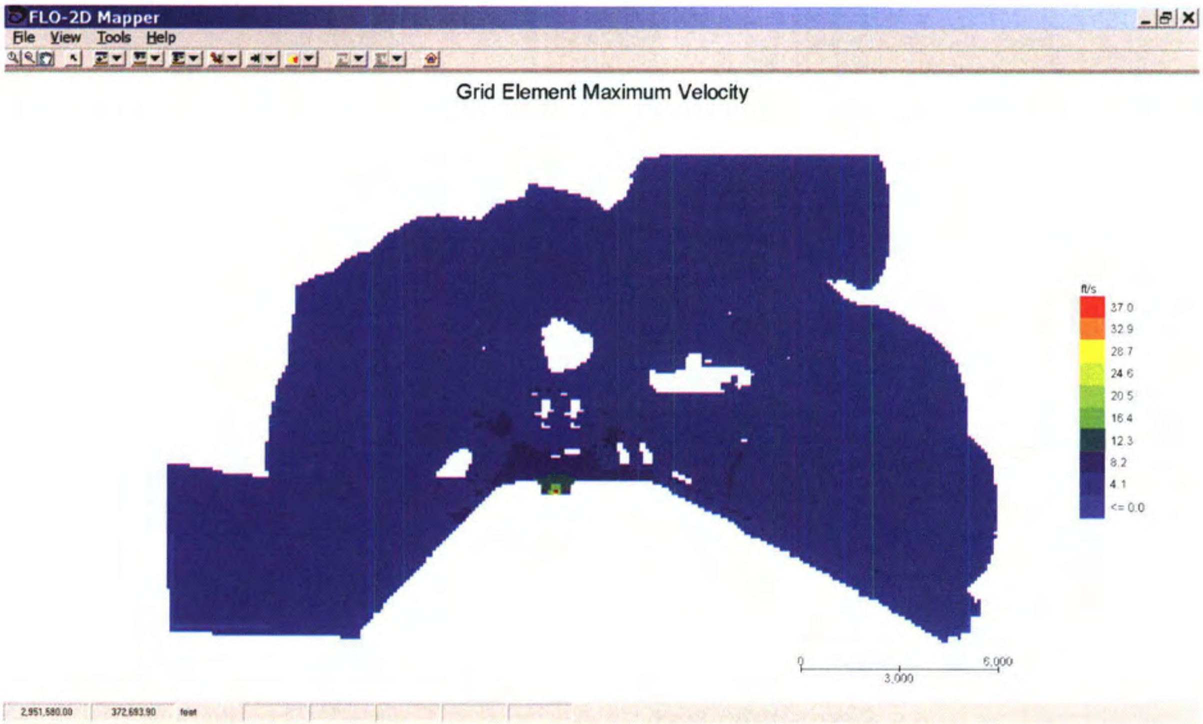


Figure 6.6-6. An example of the FLO-2D simulated maximum flood velocity map.

6.6.2 FLO-2D Model Simulation

As in the case of FLDWAV sensitivity runs, the author performed a simulation of MCR FLO-2D model for different breach widths and breach peak times. The purpose of this simulation is to set up the relation of breach scenario (characterized by breach width and peak time) and the resulting DBF level. The ranges of breach widths and peak times were determined based on the breach parameters estimated using empirical breach equations. The breach parameters were discretized (3 breach widths and 6 breach times), making a total of 18 runs. In order to monitor the simulated peak flood levels around the Units 3&4 power block area, the following 6 model nodes (node numbers) were chosen as:

17297: South of the Unit 4 Reactor Building
17553: South of the Unit 4 Service Building
18319: South of the Unit 4 UHS
17306: South of the Unit 3 Reactor Building
17562: South of the Unit 3 Service Building
18328: South of the Unit 3 UHS

On each run, the peak flood level during the simulation period (usually 5 hours) at each selected node was read from the FLO-2D output file. These peak flood level values are tabulated on the table below, and the maximum flood level (so called the DBF level) was determined from the peak flood levels at 6 nodal points. In all runs, cell number 18328 (south of the Units 3 UHS) gives the highest flood level.

Table 6.6-1. The result of simulated peak flood levels simulated by the MCR FLO-2D with different breach peak outflows and times.

Run ID	Breach width	Tp-Input	Qp	Tp	Peak Flood Level for different Cell ID						Max flood level
					17297	17553	18319	17306	17562	18328	
	ft	hr	Kcfs	hr	ft	ft	ft	ft	ft	ft	ft
b500t05	500	0.5	285	0.5	42.78	43.27	44.11	43.74	43.57	44.17	44.17
b500t10	500	1.0	277	1	42.63	43.14	43.92	43.60	43.47	44.03	44.03
b500t15	500	1.5	265	1.5	42.50	42.98	43.72	43.43	43.29	43.84	43.84
b500t20	500	2.0	241	2	42.03	42.53	43.26	42.97	42.83	43.34	43.34
b500t23	500	2.5	210	2.375	41.48	41.96	42.62	42.39	42.23	42.74	42.74
b500t27	500	3.0	189	2.7	41.01	41.47	42.11	41.89	41.74	42.19	42.19
b500t05	750	0.5	415	0.5	44.82	45.40	46.36	45.90	45.75	46.46	46.46
b500t10	750	1.0	391	1	44.62	45.12	46.06	45.62	45.47	46.19	46.19
b500t15	750	1.5	331	1.5	43.70	44.24	45.07	44.72	44.57	45.24	45.24
b500t20	750	2.0	280	1.8	42.85	43.37	44.17	43.83	43.68	44.29	44.29
b500t23	750	2.5	244	2.125	42.19	42.7	43.45	43.15	43.02	43.56	43.56
b500t27	750	3.0	219	2.4	41.67	42.16	42.87	42.60	42.44	42.93	42.93
b500t05	1000	0.5	539	0.5	46.52	47.30	48.28	47.70	47.56	48.36	48.36
b500t10	1000	1.0	466	1	45.64	46.25	47.31	46.87	46.68	47.47	47.47
b500t15	1000	1.5	369	1.35	44.35	44.88	45.80	45.38	45.29	45.95	45.95

b500t20	1000	2.0	309	1.6	43.36	43.89	44.70	44.37	44.21	44.85	44.85
b500t23	1000	2.5	271	1.875	42.74	43.23	44.01	43.71	43.54	44.15	44.15
b500t27	1000	3.0	241	2.1	42.09	42.59	43.32	43.02	42.89	43.42	43.42

6.6.3 Relation between DBF Level and Breach Parameters

Using the resulting DBF levels on the table above, the author performed a multiple linear regression analysis to obtain a regression equation to predict DBF level. The independent variable of the regression is DBF level at the Units 3 and 4 area and the dependent variables are peak breach outflow and peak time. The purpose of this regression analysis is to predict DBF level easily without re-simulating the MCR FLO-2D model that requires a substantial run-time. The resulting regression equation is as following:

$$H = 39.26042+0.01757*Qp/1000-0.03168*Tp, \text{ with } R^2=0.9895 \qquad \text{(Equ. 6.6-1)}$$

where

H is the DBF level at the Units 3&4 area in ft MSL
Qp is the peak flow in cfs, and
Tp is the peak time in hour.

The regression to predict DBF level is quite good as the regression statistics shown (refer to Table 6.6-2 and Figure 6.6-7). Qp is more sensitive to the DBF level than Tp for the possible ranges of these breach parameters. To estimate DBF levels, this equation needs peak breach outflow rate and peak time. These parameters could be estimated from empirical equations or by simulating BREACH, FLDWAV, HEC-RAS, or other breach models.

Table 6.6-2. Regression statics for predicting DBF level using peak outflow and time.

Regression Statistics					
Multiple R	0.994732				
R Square	0.989492				
Adjusted R Square	0.988091				
Standard Error	0.183661				
Observations	18				
ANOVA					
	df	*SS*	*MS*	*F*	*Significance F*
Regression	2	47.64701	23.8235	706.2698	1.44968E-15
Residual	15	0.505972	0.033731		
Total	17	48.15298			

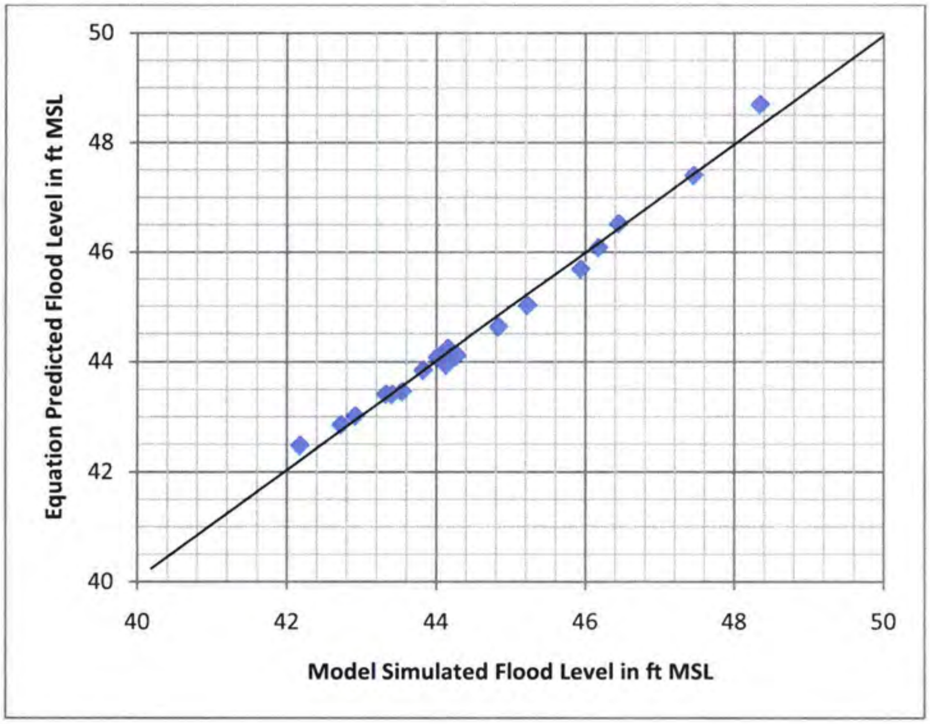


Figure 6.6-7. Linear relationship between the FLO-2D simulated DBF level and predicted one by a multiple linear regression equation.

6.6.4 Validation of FLO-2D Simulations

Direct validation of the MCR FLO-2D model is not possible because there is no actual MCR breach data. However, the model can be validated indirectly using the result of different flood routing models. For MCR breach flood routing, STP used the Delft3D model in the FSAR Revision 0 and the RMA2 model in the FSAR revision 2. The author compared the DBF level obtained by the FLO-2D simulation with those two simulations. Two MCR breach scenarios are used in the comparison: scenario for (B=417 ft, Tf=1.7 hr), and scenario for (B=4757ft, Tf=2.44 he).

STP obtained the maximum flood level of 38.9 ft MSL using the RMA2 model, then determined the DBF level of 40 ft MSL considering a margin for wind wave setup. The author simulated the FLO-2D model with the STP's breach scenario, but obtained the maximum flood level of about 40 ft MSL (without wind setup) which is about 1 foot higher than RMA2 result. The author tried to reduce the FLO-2D DBF level estimates to that of RMA2 by changing n-value, land surface elevations, structure configuration, and breach location and nodes in the model. However, the effort was fruitless nor finding any error in the MCR FLO-2D model. It is author's opinion that STP's RMA2 model could be wrong because they used the unexplained sump zone around the proposed power block area. However it is impossible for the author to check the STP's RMA2

model because the RMA2 model is not available to the staff. Instead of further efforts to refine the MCR FLO-2D model, the author attributed the one feet difference as a modeling uncertainty.

STP provided in the FSAR Revision 0 a table listing the DBF levels estimated by the Delft3D model for different MCR breach widths - They are plotted in Figure 6.6-8. Delft3D-FLOW is a multidimensional hydrodynamic and transport numerical model. This model can simulate unsteady flow and transport phenomena that result from tidal and meteorological forcing on a rectilinear or a curvilinear boundary-fitted grid system. The model solves the Navier-Stokes equations for incompressible fluid using the shallow water and the Boussinesq assumptions. The result of the Delft3D simulation for the STP breach scenario matches that by the RMA2 simulation even though the breach time for the Delft3D simulation is unknown. For the breach width of 4757 ft, FLO-2D and Delft3D gives the DBF levels of 47 ft MSL and 47.7 ft MSL, respectively.

Based on the above comparison results, it was concluded that within the model uncertainty 0f 1 foot the MCR FLO-2D provides reasonable DBF level estimates for the range of possible breach parameters.

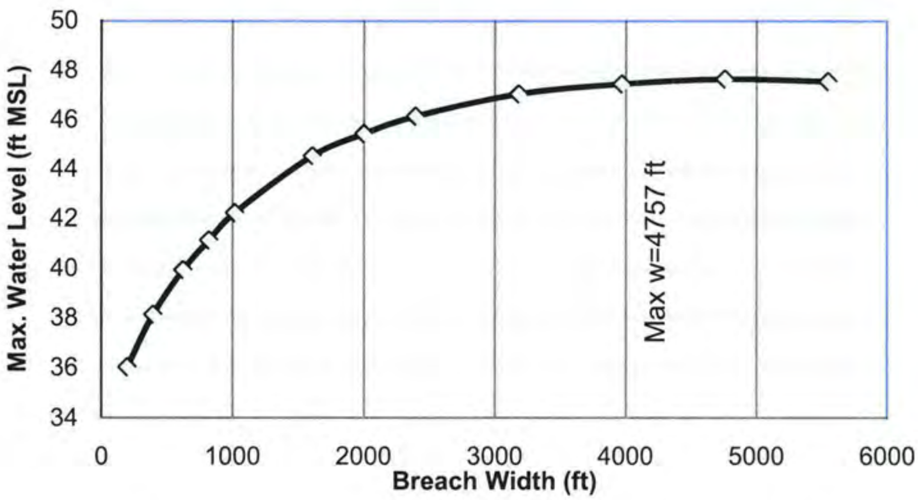


Figure 6.6-8. The relation between MCR breach width and the DBF level estimated by the STP using the Delft3D model (Based on the STP FSAR Revion 0 data).

6.6.5 Sensitivity Analysis of FLO-2D Simulation

The author performed a comprehensive sensitivity analysis for the MCR FLO-2D model. The objectives of this sensitivity analysis were to determine the effects of input variable on the DBF levels and to assure the confidence for the inputs and parameters used in FLO-2D modeling. The following input parameters were used in the sensitivity analysis:

- Model outflow boundary

- Grid size
- Manning's n-value
- Land surface elevations, especially near the site.
- Elevations for drainage ditches, Little Robin Slough, and FM521 Highway
- Location and number of the model inflow (breach outflow) nodes
- Existing and proposed structures, and
- Others

This report does not present a detailed result of these sensitivity runs because most of them are not sensitive to the DBF level compared to those for the breach width and time - The input and output files are available in the electronic format. The following is the major findings from this sensitivity test:

- Most of the cases, the model is not sensitive to the above parameters. For instance, the sensitivity analysis of Manning's n-values range from 0.02 to 0.08 (0.04 is used in the model) shows that the change of the corresponding DBF levels is only 0.34 feet which is substantially smaller than that of breach parameters. This is because the FLO-2D model uses corrected n-values based on the flow conditions and flow depths in each cells.

- The sensitivity runs for drainage ditch elevation show that they are not sensitive on the DBF levels even though they affecting the flood level at the beginning of the flood. This is because the capacities of ditches or local streams are too small to control the DBF floods.

- Also, the sensitivity results reveals that the DBL levels simulated by the FLO-2D is not sensitive to the change of the elevations near the plant area, exact locations of facilities, or breach outflow nodal conditions.

6.6.6 Estimation of DBF Levels for Different Scenarios

The author postulated 8 conservative MCR breach scenarios based on the sensitivity analyses using the BREACH, FLDWAV, and FLO-2D models. The descriptions of each scenario are followed:

- MLM-B-D10: Use the MLM breach width equation with **10 ft** scour hole depth; use the MLM breach time equation; estimate Qp and Tp from 6.5-1 and 6.5-2, respectively; n-value used.

- MLM-B-D20: Use the MLM breach width equation with **20 ft** scour hole; use the MLM breach time equation; estimate Qp and Tp from 6.5-1 and 6.5-2, respectively; n-value used.

- MLM Qp: Use the MLM Qp equation; use the MLM Tf for Tp, no scour hole, no n-value.

- Avg Qp: Same as above but use an average Qp of 251 kcfs.

- RUN1: corrected SER scenario with tailwater cross section and max levee length of 3000 ft}. Note: MCR BREACH gives a runtime error if width is greater than 3000 ft.

- RUN2: RUN 1 with a 10-ft scour hole.

- RUN23: RUN 1 with a 15ft scour hole.

- RUN24: RUN 1 with a 20-ft scour hole.

Once Qp and Tp were estimated from either empirical equations or breach models, DBF levels are computed using Equation 6.6-1. The table below lists the breach parameters as well as estimated DBF levels for each scenario. The result of this estimation indicates that the reasonably conservative DBF levels at the proposed Units 3 & 4 is about 45 ft MSL considering a margin for wind wave setup.

The DBF level estimates in this table are based on the following three different approaches:

- MPM breach width equation, FLDWAV, and FIO-2D
- MLM peak flow, FLO-2D
- BREACH, FLO-2D

For the BREACH model, n-value of 0.075 is used. The DBF level estimates for the first four scenarios were not relied on the Manning's n-value, but use different empirical equations and FLDWAV. All these approaches produced very consistent DBF estimates.

Table 6.6-3. DBF levels estimated by Equation 6.6-1 with various MCR breach scenarios.

Run ID	B (ft)	Tf (hr)	Qp (kcfs)	Tp (hr)	DBF level H (ft MSL)	Remark
MLM-B-D10	1047	2.54	309	1.87	44.63	Qp &Tp from FLDWAV,10 ft hole
MLM-B-D20	745	2.54	280	2.12	44.11	Qp &Tp from FLDWAV, 20 ft hole
MLM Qp	-	-	217	2.54	42.99	Qp &Tp from FLDWAV, 0 ft hole
Avg Qp	-	-	251	2.54	43.59	Qp &Tp from FLDWAV, 0 ft hole
RUN1	934	-	194	3.29	42.56	Qp &Tp from BREACH, 0 ft hole
RUN2	633	-	269	2.12	43.92	Qp &Tp from BREACH, 10 ft hole
RUN23	516	-	271	1.8	43.96	Qp &Tp from BREACH, 15 ft hole
RUN24	433	-	267	1.64	43.90	Qp &Tp from BREACH, 20 ft hole

6.6.7 Very Extreme MCR Breach Scenarios

The breach parameter estimates in Subsection 6.3 are based on the best-fitted linear regression line shown (dot lines in Figure 6.6-9). The problem in using the best-fitted line is that about a half of historical breach records would exceed the best-fit estimate – This expected value result in an underestimation. If an extreme conservatism is of concern, one may use a bounding (dashed envelop) line equations. On this regard, the following three extreme scenarios were postulated:

- MLM-B-D0: Use MLM breach width equation with no scour hole, then use FLDWAV to estimate Qp and Tp. It will end up a wider breach width than one with scour hole.
- Extreme B: Use a STP postulated breach width of 4745 ft in the FSAR Revision 0, then use FLDWAV to estimate Qp and Tp. This may be the case where a bounding MLM breach width equation is used.
- High Qp: Use an average of 5 high Qp's estimated using empirical equations (Qp=336 kcfs).

As before, DBF levels were computed by Equation 6.6-1 with estimated Qp and Tp. The table below lists the breach parameters as well as estimated DBF levels for each scenario. The result of this estimation shows that the extremely conservative DBF level would be about 47 ft MSL, where as FSAR Revision 0 DBF level estimate based on the simulation of the Delft3D model was is 47.6 ft MSL.

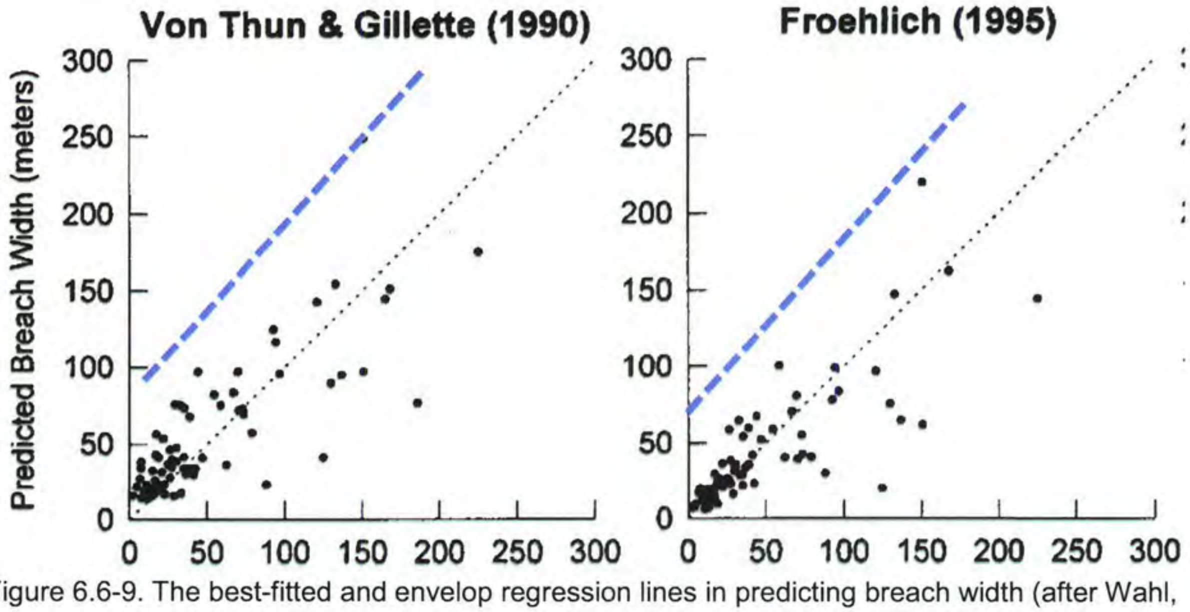


Figure 6.6-9. The best-fitted and envelop regression lines in predicting breach width (after Wahl, 1998).

Table 6.6-4. DBF levels estimated by Equation 6.6-1 with extremely conservative MCR breach scenarios.

Run ID	B (ft)	Tf (hr)	Qp (kcfs)	Tp (hr)	DBF level H (ft MSL)	Remark
MLM-B-D0	1736	2.54	380	1.55	45.89	0 ft hole
Extreme B	4745	2.54	430	1.3	46.77	0 ft hole
High Qp	-	1.7	336	1.3	45.12	MLM envelop Tf

6.7 Frequency of Occurrence of MCR Breach

FSAR 2.4 dam breach analysis is based on a deterministic approach that does not consider the condition of the levee or frequency of failure occurrence. However, this section includes an investigation of the probability of the MCR breach failure as referencing information. Generic dam failure frequency which is mainly based on small or medium size dams (length of less than 2000 ft) cannot be applicable to MRC breach. The above generic failure probability cannot be used because a total MCR levee length is 12.6 miles. The MCR breach failure frequency could be higher than the generic one. This report uses the following three sources of information:

- Foster et al. (2000: Table 5) said that an average annual frequency of embankment levee failure (earth-fill levee with filter) due to piping is about $1.89\text{x}10^{-4}$/year for first 5 years of operation and $0.37\text{x}10^{-4}$ /year after 5 years of operation.

- The frequency of levee failures for the California Delta Levee System (a total levee length of 911 miles) is about $1.18\text{x}10^{-4}$/year/levee mile.

- The frequency of the Suisun Marsh levee (a total of 75 miles) is about $4.76\text{x}10^{4}$/year/levee mile.

In summary, the estimated frequency of MCR levee failure is an order of **10^{-4}/year.** However, this estimate needs to be corrected considering the length and condition of the MCR levee.

7 Conclusion

The author of this report performed a re-analysis of the main cooling reservoir (MCR) breach flood as part of reviewing the South Texas Project combined license application. The result of the re-analysis proves that both STP's and PNNL's MCR breach flood analyses are inaccurate and non-conservative> they underestimated the MCR breach parameters (breach width and peak outflow rate) and the resulting design basis flood level. The author's re-analysis reveals that the reasonably conservative design basis flood level would be about 45 ft mean sea level, or it could exceed about 47 ft mean sea level if a very conservative MCR breach scenario is considered. Because the design basis flood level is critical in safety-related structural designs and flood protection, the author recommends revising the dam breach sections of FSAR and the current version of the SER using a correct and conservative MCR breach scenario.

References

- Acrement G. J. and V. R. Schneider, 1984, Guide for Selecting Manning's Roughness Coefficients for Natural Channel and Flood Plains, USGS Water Supply Paper 2339.

- Baecher, G.B., M. Elisabeth, and R. D., Neufville, 1980, "Risk of Dam Failure in Benefit-Cost Analysis, Water Resources Research, Vol. 16, No. 3, Pages 449-456.

- Brunner, G. W., 2007, Using HEC-RAS to Perform a Dam Break Analysis, a paper distributed at the FY07 Training for Hydrology and Hydraulics for Dam Dafety Studies by USACE/ HEC, Davis, CA.

- Chauhan, S.S., D.S. Bowles, and L.R. Anderson, 2004, ASDSO Proceeding of Dam Safety, "Do Current Breach Parameter Estimation Techniques Provide Reasonable Estimates for Use in Breach Modeling?" 2004 ASDSO Proceeding of Dam Safety, Utah State University and RAC Engineers & Economics, Logan, UT.

- Chow, V.T. 1959. Open Channel Hydraulics. McGraw-Hill, New York.

- FLD-2D Software, Inc., 2006, FLO-2D User Manual, Version 2006.01.

- Foster, M., R. Fell, and M. Spannagle, 2000, "The Statistics of Embankment Dam Failures and Accidents", Can. Geotech. J. 37:1000-1024.

- Fread, D.L., 1988, The NWS-DMABRK Model, Quick User Guide, Revision 4, 1991, Hydrologic Research Laboratory, Office of Hydrology, National Weather Service, NOAA

- Fread, D.L. 1991. BREACH: An Erosion Model for Earthen Dam Failures. Hydrologic Research

- Fread, D.L., 1988 (revised in 1991), BREACH, An Erosion Model for Earthen Dam Failures, National Weather Service, National Oceanic and Atmospheric Administration, Silver Spring, MD.

- Fread, D.L., 1993, NWS FLDWAV Model: The Replacement of DAMBRK for Dam-Break Flood Prediction, Dam Safety '93, Proceeding of the 10th Annual ASDSO Conference, Kansas City, Missouri.

- Fread, D.L., 2000. NWS FLDWAV Model: Theoretical Description," Hydrologic Research Laboratory, Office of Hydrology, National Weather Service, NOAA (Revision 4 1991).

- Froehlich, D. L. 1995a. Embankment dam breach parameters revisited. Water Resources Engineering, Proceedings of the 1995 ASCE Conference on Water Resources Engineering, New York, 887–891.

- Froehlich, D. C. 1995b. Peak outflow from breached embankment dam. Journal of Water Resources Planning and Management, American Society of Civil Engineers, 121(1), 90–97.

- Gee, D.M., 2008, Comparison of Dam Breach Parameter Estimators, HEC Report, U.S. Corps of Engineers Hydrologic Engineering Center, Davis, CA.

- Gee, D.M., 2010, Use of Breach Process Models to Estimate HEC-RAS Dam Break Parameters, 2nd Joint Federal Interagency Conference, Las Vegas, NV.

- Hydrologic Engineering Center (HEC), 2008, HEC-RASRiver Analysis Sytem, User's Manual, Version 4.0, USACE, Davis, CA.

- MacDonald, T. C., and Langridge-Monopolis, J. 1984. Breaching characteristics of dam failures. Journal of Hydraulic Engineering, 110(5), 567–586.

- Nagy, L., 2006, Estimating Dyke Breach Length from Historical Data, Periodics Polytechnica Ser. Civ. Eng. Vol 50, No. 2, pp125-139.

- Pacific Northwest National Laboratory, 2011, Calculation Worksheet: Confirm MCR embankment breach flood discharge and its sensitivity to breach parameter, submitted to NRC as a supplemental document for SER, Richmond, WA.

- SFWMD (South Florida Water Management District), 1980, Interim Final Draft Report on Embankment Failure Florida Power $ Light Company Martin Plant Cooling Reservoir, SFWMD, West Palm Beach, FL.

- STP (South Texas Nuclear Operating Company), 2007, South Texas Combined License Application, Revision 0, Part 2, Final Safety Analysis Report, Bay City, TX.

- STP (South Texas Nuclear Operating Company), 2009, South Texas Combined License Application, Revision 2, Part 2, Final Safety Analysis Report, Bay City, TX.

- STPEGS, 2006, Updated Final Safety Analysis Report (UFSAR for Units 1 & 2, Revision 13.

- U.S. Army Corps of Engineers, 2005, User's Guide to RMA2 WES, Version 4.5, Coastal and Hydraulics Laboratory, Waterway Experiment Station, Engineering Research and Development Center, Vicksburg, MS.

- U.S. Army Corps of Engineers, 2006, "Engineering and Design: Reliability Analysis and Risk Assessment for Seepage and Slope Stability Failure Modes for Embankment Dams", Department of Army, U.S. Army Corps of Engineers, ETL 1110-2-561, Washington, D.C.

- United States Geological Survey 2009. Water Resources of Illinois: n-Values Project. URL: http://il.water.usgs.gov/proj/nvalues/equations.shtml?equation=05-strickler.

* U.S. Bureau of Reclamation 1982. Guidelines for defining inundated areas downstream from Bureau of Reclamation dams, Reclamation Planning Instruction No. 82-11, U.S. Department of the Interior, Bureau of Reclamation, Denver, Colorado, 25p.
* U.S. Bureau of Reclamation 1988. Downstream hazard classification guidelines. ACER Technical Memorandum No. 11, U.S. Department of the Interior, Bureau of Reclamation, Denver, Colorado, 57p.
* U.S. Bureau of Reclamation, 1998, Prediction of Embankment Dam Breach Parameters: A Literature Review and Needs Assessment", DSO-98-004, Dam Safety Research Report, U.S. Department of the Interior, Bureau of Reclamation, Dam Safety Office.
* URS, 2007, "Technical Memorandum Delta Risk management Strategy (DRMS) Phase 1: Topical Area Levee Vulnerability Draft 2, URS Corporation/J.R. Benjamin & Associates, Inc., Oakland, CA.
* Von Thum, J.L. and D.R. Gillette, 1990, Guidance on Breach Parameters, unpublished internal document, U.S. Bureau of Reclamation, Denver, CO.
* Wahl, T.L. 2004. Uncertainty of Predictions of Embankment Dam Breach Parameters. Journal of Hydrology, 130(5), 389-397.
* Wahl, T.L., 1997, Predicting Embankment Dam Breach Parameters – A Needs Assessment, XXVIIth IAHR Congress, San Francisco, CA.
* Wahl, Tony L. , Gregory J. Hanson, Jean-Robert Courivaud, Mark W. Morris, René Kahawita, Jeffrey T. McClenathan, and D. Michael Gee, 2008. Development of Next-Generation Embankment Dam Breach Models. U.S. Society on Dams, 2008 Annual Meeting and Conference, April 28-May 2, Portland, Oregon, [online paper].
* Zhang X. and Wang, G, 2001, Flow Analysis and Scoring Hole Computation of Dyke-Breach," Proceeding of the International Association for hydraulic Researchers XXIX Congress, Theme E. Tsinghua University, Beijing, China.

Attachment 11c: Enclosure 2: PNNL's Calculation Package/Worksheet, commented on by H. Ahn June 20, 2011

Enclosure 2: PNNL's Calculation Package - Commented and corrected by Dr. Hosung Ahn

Calculation Worksheet

Title: Confirm MCR embankment breach flood discharge and its sensitivity to breach parameters			**Project**: PNNL Project #
Prepared By: LF Hibler, R Prasad, and Y Yuan	**Date**: 3/16/2011	**Reviewed By**: SA Breithaupt	**Date**:

- **Purpose**:
  - Perform estimation of main cooling reservoir (MCR) embankment breach parameters using available and applicable empirical equations, the National Weather Service (NWS) BREACH model, and evaluate FLDWAV runs to confirm the applicant's assessment of the MCR embankment failure flood at the STP site. Perform sensitivity studies on the BREACH model.

- **Methods Used**:
  - The applicant used a combination of the MacDonald and Langridge-Monopolis (1984) (MLM) and Froehlich (1995a, b) approaches to develop estimates of the MCR breach parameters.
  - We evaluated the applicant's estimates of MCR embankment breach parameters using several approaches. These approaches included:
    - MLM (1984) empirical approach
    - Froehlich (1995a, b) empirical approach
    - NWS BREACH model
    - FLDWAV model

- **Assumptions & Constraints**:
  - **Assumption 1**: The predicted volume of eroded material (Ve*) in the MLM approach is the volume of embankment material, not the volume of solid particles.
  - **Verification of Assumption 1**: To verify Assumption 1, we used the Apishapa Dam data reported by MacDonald and Langridge-Monopolis (1984). The volume of the breach (V_t) was computed based on the breach height (H), top width (T_w), breach width at top (B_t), upstream and downstream side slopes of the embankment (Se1 and Se2). The computation was repeated except that breach width at bottom (B_b) rather than the breach width at top and side slopes of the breach (S_{b1} and S_{b2}) were used to estimated the volume (Vb). The two estimated volumes, Vt and Vb, were averaged ($V_a=(V_t+V_b)/2$) and the average volume was compared with the volume (V_e*) reported in by MacDonald and Langridge-Monopolis (1984). A relative difference was computed (RD=100*(V_e*-V_a)/V_e*).

    MacDonald and Langridge-Monopolis (1984) reported the following values:

    V_e* = 291,000 cubic yards
    H = 100 ft
    B_t = 320 ft
    T_w = 16 ft

S_{e1} = 1//3
S_{e2} = 1/2
S_{b1} = 6.7
S_{b2} = 2.9.

We calculated

$V_t = B_t [T_w H + 0.5 H^2 (1/S_{e1} + 1/S_{e2})]/27$ = 315,259 cubic yards
$V_b = (B_t - H/S_{b1} - H/S_{b2}) [T_w H + 0.5 H^2 (1/S_{e1} + 1/S_{e2})]/27$ = 266,583 cubic yards
$V_a = 0.5 (V_t + V_b)$ = 290,921 cubic yards
$RD = 100 (V_e^* - V_a)/V_e^*$ = 0.027 percent.

Because the relative difference is small, Assumption 1 is confirmed. The MLM-predicted volumes should be interpreted as embankment or dam volumes rather than solid material volumes.

- **Calculation Basis**:
  - We used data provided by South Texas Nuclear Operating Company (STPNOC) in Units 3 and 4 Final Safety Analysis Report (FSAR), a part of the Combined License (COL) application to the U.S. Nuclear Regulatory Commission, responses to various Request for Additional Information (RAI), and technical reports prepared by STPNOC's contractors.

    The methods used in this calculation worksheet are described in the references listed below. We describe our assumptions, verification of these assumptions where appropriate, and steps used in these calculations.

    We ran the NWS BREACH model using the input file provided by STPNOC. We were able to reproduce the results of the NWS BREACH model reported by STPNOC in response to RAI 02.04.04-14 (STPNOC 2010). Several variations of BREACH parameters were used to investigate the sensitivity of model predictions.

- **Calculation Input**:
  - **Estimation of MCR embankment breach parameters**:
    - NWS BREACH model
    - Empirical approaches (see Wahl 2004)

  - **Estimation of discharge hydrograph following the MCR embankment breach**:
    - Based on the BREACH sensitivity analysis, we selected a conservative set of parameters that is expected to result in conservative predictions of breach size and peak discharge.

  - **Sensitivity analysis for embankment breach parameters**:
    - NWS BREACH model
      - Sensitivity of the model to elevation at which piping failure commences, Z_p (ft)
      - To evaluate the sensitivity of the model to Z_p, we used the following values: Z_p = { 29, 30, 31, 32, 33, 34, 35, 36, 37, 38, 39, 40, 41, 42, 43, 44, 45 } ft
      - The applicant used a Z_p of 34 ft MSL (STPNOC 2010).

      - Sensitivity of the model to Manning's n of dam material

- To evaluate the sensitivity of the model to n, we used the following values: n = { 0.007, 0.008, 0.010, 0.015, 0.020, 0.025, 0.030, 0.040, 0.050, 0.060, 0.070, 0.080 }
- The applicant used a Manning's n value of 0.05 (STPNOC 2010). Strickler's equation for prediction of Manning's n from median grain size is (Fread, 1991)

  $n = 0.013\ (D_{50})^{0.67}$

  with D_{50} in mm. Using the applicant's D_{50} value of 0.001 mm, n is estimated as 0.0001. The applicant stated that this estimate was too low and used a value of 0.05 for Manning's n (STPNOC 2010).

  Strickler's equation is also given by USGS (2009). The USGS (2009) form of the equation is

  $n = 0.015\ (D_{50})^{(1/6)}$

  USACE (1994) also indicates that the exponent in Strickler's equation should be 1/6 or 0.167, but the units for D_{50} is ft:

  These n-values are too small - need justification.

  $n = 0.034\ (D_{50})^{(1/6)}$

  Using the three forms of Strickler's equation, we estimated the Manning's n as 0.0001 (Fread 1991 form), 0.005 (USGS 2009 form), and 0.004 (USACE 1994 form).

- Sensitivity of the model to cohesive strength of dam material, C (lb/ft^2)
  - To evaluate the sensitivity of the model to C, we used the following values: C = { 50, 100, 150, 200, 250, 300, 350, 400 } lb/ft^2
  - The applicant used a C of 300 lb/ft^2. The applicant's chosen value is the minimum of the reported undrained and drained cohesive strengths of the embankment material (Bechtel Energy Corporation 1984).

- Sensitivity of the model to internal friction angle of dam material, Ø (degrees)
  - To evaluate the sensitivity of the model to internal friction angle, we used the following values: Ø = { 10, 15, 20, 25, 30 } degrees
  - The applicant used a Ø of 20 degrees. Bechtel Energy Corporation (1984) reported values of Ø to be 16 and 20 degrees for the undrained and drained embankment material.

- Sensitivity of the model to length of dam, L (ft)
  - To evaluate the sensitivity of the model to the length of the dam, we used the following values: L = { 1000, 2000, 3000, 4000 } ft
  - The applicant used an L of 1000 ft. The east-to-west running portion of the north face of the MCR embankment is approximately 4300 ft in length.

▪ Empirical approaches
- Depth of water above breach invert at time of failure, h_w = 21.9 ft (6.68 m)
- Volume of water stored above breach invert at time of failure, V_w = 152,700 ac-ft (188,352,670.9 m^3)

- Height of breach, h_b = 37 ft (11.28 m)
- Overtopping multiplier, K_o = 1.4 for overtopping, 1.0 for piping

- **References**:

  - **Bechtel Energy Corporation 1984**. Evaluation of Strength Parameters and Stability Main Cooling Reservoir Embankment. Volume 1 Report. Harza Engineering Company, Houston, Texas.
  - **Chow, V.T. 1959**. Open Channel Hydraulics. McGraw-Hill, New York.
  - **Fread, D.L. 1991**. BREACH: An erosion model for earthern dam failures. Hydrologic Research Laboratory, Office of Hydrology, National Weather Service, U.S. National Oceanic and Atmospheric Agency, Silver Spring, Maryland, July, 1991.
  - **Froehlich, D. C. 1995a**. Embankment dam breach parameters revisited. Water Resources Engineering, Proceedings of the 1995 ASCE Conference on Water Resources Engineering, New York, 887–891.
  - **Froehlich, D. C. 1995b**. Peak outflow from breached embankment dam. Journal of Water Resources Planning and Management, American Society of Civil Engineers, 121(1), 90–97.
  - **MacDonald, T. C., and Langridge-Monopolis, J. 1984**. Breaching characteristics of dam failures. Journal of Hydraulic Engineering, 110(5), 567–586.
  - **Merz, B. and A.H. Thieken 2005**. Separating natural and epistemic uncertainty in flood frequency analysis. Joural of Hydrology, 309(1-4), 114-132.
  - **STPNOC 2010**. Letter from Scott Head to NRC Document Control Desk, U7-C-STP-NRC-100241, November 22, 2010.
  - **U.S. Army Corps of Engineers 1994**. Hydraulic Design of Flood Control Channels. Engineer Manual EM 1110-2-1601, Washington DC.
  - **U.S. Bureau of Reclamation 1982**. Guidelines for defining inundated areas downstream from Bureau of Reclamation dams, Reclamation Planning Instruction No. 82-11, U.S. Department of the Interior, Bureau of Reclamation, Denver, Colorado, 25p.
  - **U.S. Bureau of Reclamation 1988**. Downstream hazard classification guidelines. ACER Technical Memorandum No. 11, U.S. Department of the Interior, Bureau of Reclamation, Denver, Colorado, 57p.
  - **United States Geological Survey 2009**. Water Resources of Illinois: n-Values Project. URL: http://il.water.usgs.gov/proj/nvalues/equations.shtml?equation=05-strickler.
  - **Wahl, T.L. 2004**. Uncertainty of Predictions of Embankment Dam Breach Parameters. Journal of Hydrology, 130(5), 389-397.

- **Units**:
  - Some of the estimation equations and model input data used SI units. When original data were available in English units, we used equivalent values converted to SI units for these inputs. The final results from the calculations are reported in English units after conversion.

- **Results**:
  - **Sensitivity analysis for embankment breach parameters**:
    - NWS BREACH model
      - Sensitivity of the model predictions to elevation at which piping failure commences, Z_p (ft). The model predictions are fairly insensitive to Z_p. At Z_p of 30 ft and lower, the model run did not complete because of a mathematical error, probably a result of Z_p being too close to the bottom of the reservoir. At Z_p of 46 ft and higher, which is very close to the top of the initial water surface elevation in

the reservoir, it appears the breach did not develop fully to erode a large portion of the embankment.

Z_p (ft MSL)	Q_p (cfs)	B_t at Peak Flow (ft)	Bb at Peak Flow (ft)	T_p (hr)
29	--	--	--	--
30	--	--	--	--
31	83,261	412.5	360.7	6.1
32	83,239	412.4	360.6	6.2
33	83,216	412.9	361.1	6.2
34	83,197	412.4	360.6	6.3
35	83,176	412.1	360.3	6.3
36	83,152	412.3	360.5	6.4
37	83,175	412.1	360.3	6.4
38	83,200	412.4	360.6	6.4
39	83,074	411.1	359.3	6.7
40	83,089	411.3	359.5	6.7
41	83,101	411.3	359.5	6.8
42	83,110	412.3	360.5	6.9
43	83,123	411.8	359.9	7.1
44	83,137	411.7	359.9	7.4
45	83,150	413.1	361.3	12.7
~~46~~	~~416~~	~~4.9~~	~~4.9~~	~~0.0~~
~~47~~	~~238~~	~~3.9~~	~~3.9~~	~~0.0~~

These are model errors. Should delete them

- Sensitivity of the model predictions to Manning's n of dam material. The model predictions are very sensitive to Manning's n; however, it should be noted that the range of variation of Manning's n covers three orders of magnitude. The applicant, using Strickler's equation obtained an estimate of 0.0001 for Manning's n with a D_{50} of 0.001 mm. Using the three forms of Strickler's equation presented above, we estimated the Manning's n as 0.0001 (Fread 1991 form), 0.005

(USGS 2009 form), and 0.004 (USACE 1994 form). For comparison, recommended lowest Manning's n for smooth brass, Lucite, and glass channels flowing partially full are at least nearly two times greater at 0.009, 0.008, and 0.009, respectively (Chow 1959). To estimate the sensitivy of model predictions we varied n from 0.0001 to 0.1, a range of three orders in magnitude, to accommodate the applicant's estimate of n, reasonable values of n reported in literature, and a conservatively selected upper range.

PNNL set up the BREACH model incorrectly so that breach width and outflow are constricted by small levee width and tailwater channel width.
Red ones are corrected values with levee width and tailwater section of 3000 ft

n	Q_p (cfs)	B_t at Peak Flow (ft)	B_b at Peak Flow (ft)	T_p (hr)
0.0001	445	3.7	3.7	1.25
0.001	2	0.3	0.3	1.25
0.005	876	5.2	5.2	1.25
0.008	4,199	11.4	11.4	1.25
0.010	8,424	16.4	16.4	1.25
0.015	15,833	110.4	58.6	23.7
0.020	22,727	144.5	92.7	19.6
0.025	30,762	183.6	131.8	15.9
0.030	~~39,776~~ 48700	~~229.7~~ 317	~~177.9~~ 206	~~13.3~~ 10.5
0.040	~~60,500~~ 73800	~~329.1~~ 438	~~277.3~~ 327	~~9.4~~ 7.55
0.050	~~83,197~~ 103000	~~412.4~~ 580	~~360.6~~ 470	~~6.3~~ 5.79
0.060	~~99,695~~ 136000	~~449.1~~ 739	~~397.3~~ 629	~~4.1~~ 4.63
0.070	~~112,518~~ 172000	~~481.2~~ 917	~~429.4~~ 806	~~2.9~~ 3.85
0.080	~~122,806~~ 207000	~~516.6~~ 1017	~~464.8~~ 906	~~2.4~~ 2.93
0.100	~~136,945~~ 252000	~~564.3~~ 1093	~~512.5~~ 983	~~1.68~~ 1.82

Sensitivity of the model predictions to cohesive strength, C, of dam material. The model predictions are no sensitive to values of C ranging from 250 to 400 lb/ft^2. As values of C were lowered from 250 lb/ft^2, peak discharge and breach width increased and the time to peak decreased. However, even with a very low cohesive strength of 50 lb/ft^2, the embankment breach width at peak flow was approximately 512 ft.

C (lb/ft^2)	Q_p (cfs)	B_t at Peak Flow (ft)	B_b at Peak Flow (ft)	T_p (hr)

50	99,628	512.3	377.4	4.35
100	92,009	460.9	364.4	4.93
150	92,009	460.9	364.4	4.93
200	87,013	460.4	363.9	6.01
250	83,197	412.4	360.6	6.25
300	83,197	412.4	360.6	6.25
350	83,197	412.4	360.6	6.25
400	83,197	412.4	360.6	6.25

- Sensitivity of the model predictions to internal friction angle, Ø, of dam material. The model predictions are only slightly sensitive to this parameter.

Ø (degrees)	Q_p (cfs)	B_t at Peak Flow (ft)	B_b at Peak Flow (ft)	T_p (hr)
10	84,571	415.9	353.8	5.76
15	84,143	413.8	357.0	5.95
20	83,197	412.4	360.6	6.25
25	82,012	403.0	355.9	6.31
30	80,959	394.6	351.9	6.36

- Sensitivity of the model predictions to length of dam, L (ft). Because the east-to-west running portion of the north face of the MCR embankment is approximately 4300 ft in length, we limited the dam length to 4000 ft. ~~Model prediction were not at all sensitive to L values ranging from 1000 to 4000 ft.~~

This conclusion is incorrect. Model predictions were very sensitive to L for the range from 400 ft to 1000 as shown on the next table.

L (ft)	Q_p	B_t at Peak Flow	B_b at Peak Flow	T_p
	207000	1000	889	2.85
1000	~~83,197~~	~~412.4~~	~~360.6~~	~~6.25~~
	207000	1017	906	2.93
2000	~~83,197~~	~~412.4~~	~~360.6~~	~~6.25~~
	207000	1017	906	2.93
3000	~~83,197~~	~~412.4~~	~~360.6~~	~~6.25~~
	207000	1017	906	2.93
4000	~~83,197~~	~~412.4~~	~~360.6~~	~~6.25~~

Sensitivity of the model predictions to length of dam, L (ft), with cohesive strength, C, set to 100 lb/ft^2, and Manning's n set to 0.06. We chose these values of the other parameters because they are expected to result in larger breach sizes. Notice that when the dam length is limiting (L = 500 ft), BREACH predicts washout of the entire embankment at the top, but predicted breach does not grow wider than the length of the embankment itself. ~~When the dam length is not limiting, model predictions are not sensitive to this parameter.~~

This is incorrect - Model predictions are sensitive if the model is set up correctly (e.g., not constricted by both breach width and tailwater section width.)

L (ft)	Q_p (cfs)	B_t at Peak Flow (ft)	B_b at Peak Flow (ft)	T_p (hr)
	114000	500	412	3.08
500	~~122,550~~	~~500.0~~	~~412.0~~	~~1.81~~
	134000	742	632	5.03
1000	~~128,954~~	~~568.6~~	~~472.2~~	~~2.28~~
~~2000~~	~~128,954~~	~~568.6~~	~~472.2~~	~~2.28~~
~~3000~~	~~128,954~~	~~568.6~~	~~472.2~~	~~2.28~~
~~4000~~	~~128,954~~	~~568.6~~	~~472.2~~	~~2.28~~

- Sensitivity of model predictions to size of tailwater cross-section. The applicant used the tailwater cross-section given in column 2 of the table below (TWCS 0). We investigated the sensitivity of the model by enlarging the tailwater cross-section given by TWCS 1-6 scenarios.

Elevation (ft)	Tailwater Cross-Section (TWCS) Scenarios								
	B_t	n	0	1	2	3	4	5	6
28	0	0.06	0	0	0	0	0	0	0
29	600	0.06	600	1000	1200	1600	2000	2400	3000
32	1200	0.06	1200	1200	1200	1600	2000	2400	3000
34	1600	0.06	1600	1600	1600	1600	2000	2400	3000
40	2000	0.06	2000	2000	2000	2000	2000	2400	3000
45	2400	0.06	2400	2400	2400	2400	2400	2400	3000
50	2800	0.06	2800	2800	2800	2800	2800	2800	3000

The predicted peak discharges, breach widths, and times-to-peak for the TWCS 1-6 scenarios are shown in the table below.

TWCS	Q_p (cfs)	B_t at Peak Flow (ft)	B_b at Peak Flow (ft)	T_p (hr)
0	83,197	412.4	360.6	6.25

Simulation for scenarios 1,2, and 3 are incorrect but were not simulated.

Predictions for 3,4, and 5 were corrected using realistic levee width and tailwater section of 3000 ft.

1	97,314	496.2	385.4	4.43
2	99,203	517.3	406.6	4.76
3	101,493	540.4	429.6	5.05
4	207000 ~~102,971~~	1017 ~~580.1~~	906 ~~469.3~~	2.93 ~~5.70~~
5	207000 ~~102,971~~	1017 ~~580.1~~	906 ~~469.3~~	2.93 ~~5.70~~
6	207000 ~~102,971~~	1017 ~~580.1~~	906 ~~469.3~~	2.93 ~~5.70~~

lection of BREACH scenarios for independent confirmatory analysis

- There is considerable uncertainty in predictions from the BREACH model. There is also considerable uncertainty in predictions from empirical approaches (Wahl 2004). Generally, uncertainty in predictions can be classified as aleatory and epistemic (Merz and Thieken 2005). Aleatory or natural uncertainty arises from inherent variability in natural processes such as randomness of soil properties. Epistemic uncertainty results from incomplete understanding of the natural system being described such as incomplete process description that does not adequately capture the behavior of the system and errors related to measuring parameters used to describe the system.

This statement is incorrect because soil property other than Manning's n is insensitive to breach parameter prediciton.

In the context of this review, the postulated MCR embankment breach occurs due to a piping failure. It has been shown (see FSAR and SER Sections 2.4S.3, 2.4S.4, 2.4S.5, and 2.4S.8) that the MCR would not overtop. The empirical approaches do not explicitly consider the failure mechanism. The BREACH model directly considers the failure mechanism; it can simulate piping and overtopping failures. Therefore, the uncertainty with respect to breach mechanism is reduced because an overtopping-induced failure is eliminated. However, uncertainty still exists in how the BREACH model simulates piping failure of an embankment. Some of this uncertainty arises from incomplete knowledge and understanding of the piping failure process and some from incomplete knowledge of the parameters used in BREACH to simulate piping failures. We have investigated the effect of Z_p, the elevation at which piping failure starts, which can be thought of as partially addressing epistemic uncertainty. Other parameters that are included in the sensitivity analyses described above address aleatory uncertainty.

This portion should be changed as "a piping or landslide failure."

- Based on the sensitivity analyses carried out above, we selected a few parameter sets to run BREACH simulations. Because the BREACH predictions are fairly insensitive to Z_p, C, and Ø, we selected the values of these parameters such that they would generally be expected to result in more conservative peak discharge and time to peak. We used Manning's n of 0.025, 0.050, and 0.075 for our BREACH simulations, because of the high sensitivity of BREACH predictions to Manning's n.

Simulation 1 { Z_p, C, Ø, n }: { 32 ft, 200 lb/ft^2, 15 degrees, 0.025 }
Simulation 2 { Z_p, C, Ø, n }: { 32 ft, 200 lb/ft^2, 15 degrees, 0.050 }
Simulation 3 { Z_p, C, Ø, n }: { 32 ft, 200 lb/ft^2, 15 degrees, 0.075 }

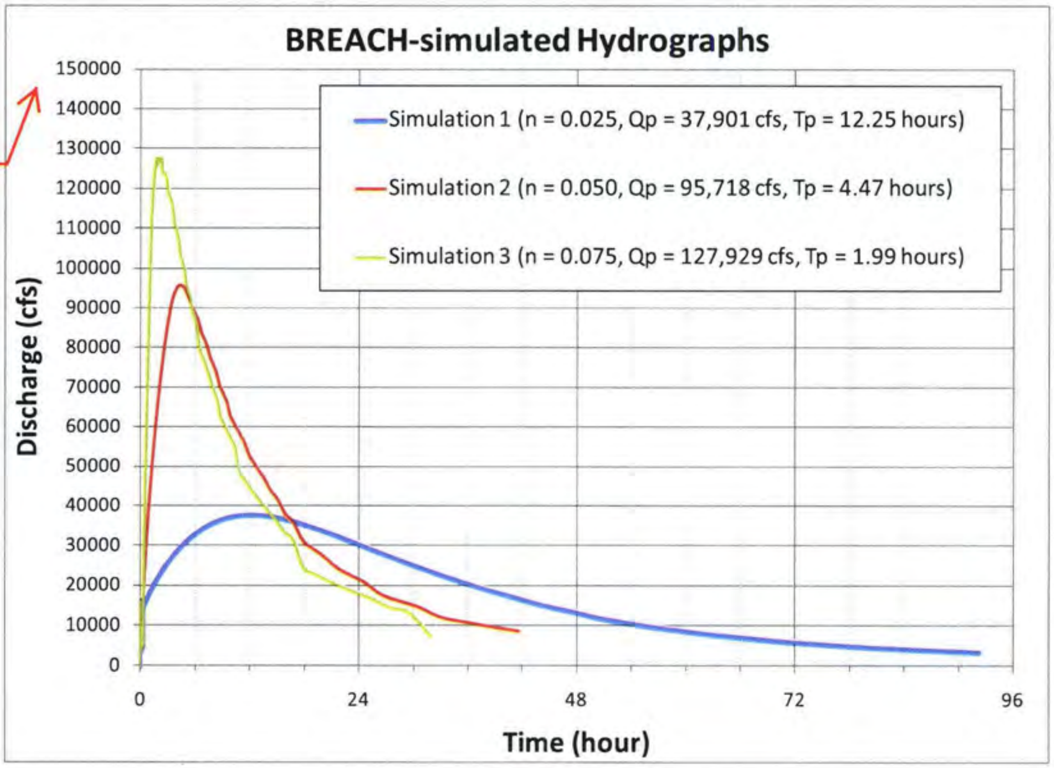
Simulation	Q_p (cfs)	B_t at Peak Flow (ft)	B_b at Peak Flow (ft)	T_p (hr)
1	37,901	263.0	152.3	12.25
2	103000 ~~95,718~~	580 ~~490.6~~	470 ~~379.8~~	5.79 ~~4.47~~
3	194000 ~~127,929~~	989 ~~574.3~~	878 ~~463.6~~	3.29 ~~1.99~~

This run uses n of 0.076 because n of 0.075 gives a BREACH model run-time error.

For simulations shown above, the breach continued to grow after peak flow was reached. For simulation 1, B_t and B_b values at 92 hours were 360.7 and 250.0 ft. For simulation 2, B_t and B_b values at 41.7 hours were 592.4 and 481.7 ft. For simulation 3, B_t and B_b values at 31.9 hours were 743.3 and 632.5 ft.

- **Outflow hydrographs following MCR embankment breach**
  - The hydrographs predicted by BREACH for the three simulations that we used are shown below.

This graph needs to be corrected with correct simulation result.



- **Comparison of MCR embankment breach hydrographs with historical dam breaches with similar characteristics**
  - We used the Reclamation dam breach database to identify historical breaches of dams that have characteristics similar to the MCR. The MCR height of water above the breach, h_w, and volume stored above the breach bottom, V_w, are 21.9 ft (6.68 m), and 152,700 acre-feet (188,352,699 m^3) respectively. The Dam Failure Database was sorted to find entries that were similar to the MCR embankment in terms of h_w

and V_w. The range of h_w considered was 15 to 50 ft (about 4 to 15 m) and the range of V_w considered was 100,000 to 300,000 acre-feet (about $1.23x10^8$ m3 to $3.70x10^8$ m^3). The database has multiple records for dam break events to allow for different estimates for the same parameter to be included. The sorting treated each record uniquely. The total number of records in the database is 410 but some records are incomplete for some of the parameters. Partial records were retained in the sort.

There were 172 records that has h_w in the range of interest. The minimum and maximum reported breach parameter values are given in the table below:

Parameter	Minimum	Maximum
Water Height above breach bottom (h_w) (m) [ft]	4.05 [13.29]	15.2 [49.87]
Peak flow (Q_p) (m3/s) [cfs]	29.4 [1038]	3115 [110005]
Final breach top width (m) [ft]	9.2 [30.18]	153.0 [502.0]
Final breach bottom width (m) [ft]	1.7 [5.58]	97.0 [318.2]
Average final breach width (m) [ft]	4.7 [15.42]	185.9 [609.9]
Breach formation time (hr)	0.25	1.5
Failure time (hr)	0.5	5.0

The database had nine entries that included volumes above the breach bottom, V_w, in the ranges of interest. Seven of these records are associated with the Teton Dam failure and the remaining two are associated with the Martin Cooling Pond failure.

Parameter	Teton	Martin Cooling Pond
Water Height above breach bottom (h_w) (m) [ft]	67.06 -83.82 [219.9 -275.0]	8.53 [27.99]
Volume of water above breach bottom (m3) [acre-ft]	$3.10x10^8$ [251,321]	$1.36x10^8$ [110,257]
Peak flow (Qp) (m3/s) [cfs]	65,120 - 65,136 [2,299,691 - 2,300,256]	3115 [110,005]
Final breach top width (m) [ft]		
Final breach bottom width (m) [ft]		
Average final breach width (m) [ft]	151 [495]	185 [607]
Breach formation time (hr)	1.25	
Failure time (hr)	4	

These historical data could be presented as a reference, however they cannot be used to justify MCR levee breach. It is highly subjective process. Empirical equations are established using several hundred dam breach data, thus we cannot simply ignore the established equations.

The only entries in the database that met both search criteria are associated with the

Martin Cooling Pond.

- **MCR embankment breach parameters estimated from empirical approaches**
  - The following empirical approaches were used. Only those approaches were used that provided estimates of all three parameters – average breach width, failure time, and peak flow.
    - Bureau of Reclamation (1982, 1988)
      - Average breach width (m), $B_{avg} = 3\ h_w$
      - Failure time (hr), $t_f = 0.011\ B_{avg}$
      - Peak flow (m³/s), $Q_p = 19.1\ h_w^{1.85}$
    - MacDonald and Langridge-Monopolis (1984)
      - Volume of eroded material (m³), $V_{er} = 0.0261\ (V_w\ h_w)^{0.769}$
      - Failure time (hr), $t_f = 0.0179\ V_{er}^{0.364}$
      - Enveloping peak flow (m³/s), $Q_p = 3.85\ (V_w\ h_w)^{0.411}$
    - Froehlich (1995a, b)
      - Average breach width (m), $B_{avg} = 0.1803\ K_o\ V_w^{0.32}\ h_b^{0.19}$
      - Failure time (hr), $t_f = 0.00254\ V_w^{0.53}\ h_b^{-0.9}$
      - Peak flow (m³/s), $Q_p = 0.607\ V_w^{0.295}\ h_w^{1.24}$

We should use peak flow equation (Qp=1.154(Vw*hw)^.44) instead of the bounding equation.

These comparison is not valid because the first 3 are overall breach width while the last 3 are peak breach width.

The table below shows the results of using empirical approaches to predict the characteristics of the MRC embankment failure. We also include the BREACH model predictions for comparison.

Approach	Average Breach Width (m [ft])	Failure Time (hr)	Peak Flow (m³/s [cfs])
Reclamation	20.0 [65.7]	0.66	5,765 (22,606) ~~4,886 [172,549]~~
MacDonald and Langridge-Monopolis	649.3 [2130.2]	1.67	6152 (217247) ~~21,151 [746,933]~~
Froehlich	127.0 [416.8]	6.98	1,765 [62,315]
BREACH Simulation 1	93.1 [305.4]	12.25	1,073 [37,901]
BREACH Simulation 2	160(525) ~~163.7 [537.1]~~	5.79 ~~4.47~~	2917(103000) ~~2,710 [95,718]~~
BREACH Simulation 3	285(934) ~~209.7 [687.9]~~	3.29 ~~1.99~~	5493(194000) ~~3,622 [127,929]~~

MLM peak flow is similar to the corrected BREACH simulation 3.

Note ~~1:~~ MacDonald and Langridge-Monopolis (1984) approach estimates the volume of material eroded by the breach. The breach width was estimated using the embankment geometry. The embankment geometry shown by Bechtel Energy Corporation (1984, Figure 27) was used to determine the area of cross-section of the MCR embankment. We estimated the volume of eroded material to be 259,479.4 m³ (9,163,427.4 ft³). The estimated corresponding average breach width was estimated by dividing the volume of eroded volume by the embankment's cross-section area and is reported above.

Note 2: Froehlich (1995a) equation for failure time uses height of the breach, h_b. The applicant, in response to RAI 2.4.4-14, used h_w in place of h_b in Froehlich's equation for failure time. The applicant's estimate of failure time is 11.1 hr (STPNOC

2010).

Note 3: As reported above, the BREACH model simulations resulted in breach widths that are somewhat larger than those predicted by the Froehlich approach but significantly smaller than the MacDonald and Langridge-Monopolis approach. The failure times in BREACH approach the MacDonald and Langridge-Monopolis prediction for a very conservative parameter set. Predicted peak flows from the BREACH model are generally in the range of that from the Froehlich approach, but significantly smaller than the MacDonald and Langridge-Monopolis approach.

- **A note on application of FLDWAV in estimation of MCR embankment breach outflow hydrograph**
  - The NWS BREACH model produces estimates of the breach growth and breach outflow (hydrograph) over time; these can be coupled to produce sediment flux over time. The growth of the breach is estimated based on hydraulics of the embankment and geotechnical parameters of the embankment material. FLDWAV could be used with prescribed timing parameters that specify breach growth such that FLDWAV-estimated discharge hydrograph and breach formation approximate those produced by BREACH. If the conceptual model for the subsequent flooding includes multiple/cascading breaches or channel networks, FLDWAV would be the appropriate model for simulating the more complex flow scenario. However, in the case of the postulated breach of the MCR embankment, a single breach is the conceptual model and therefore only BREACH is necessary to characterize the outflow event. Therefore, we did not use FLDWAV to estimate the outflow hydrograph from the MCR embankment breach.

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COMMENTS FOR THE NCP REVIEWER TO CONSIDER

See attached information:

1. Section B Summary and Recommendations by Richard Raione, December 20, 2011, 3pp.
2. STP NCP Independent Review Summary and Detailed Analyses, December 15, 2011, 40 pp.
3. Storm Surge Response to the Independent Contractor Reports, December 5, 2011, H. Jones, 5pp.
4. STP-NC-Review-Storm Surge-Irish, November 29, 2011, 12 pp.
5. STP-NC-Review-Storm Surge-Leuttich, November 22, 2011, 27 pp.
6. STP-NC-Review-Storm Surge-Resio, November 22, 2011, 9 pp.
7. STP-NC-Review-MCR Breach-Wahl, November 22, 2011, 26 pp.
8. STP-NC-Review-MCR Breach-Patev, November 30, 2011, 8 pp.
9. STP-NC-Review-MCR Breach-Beacher, November 22, 2011, December 19, 2011, 8 pp.

10a. Response to: Re-Analysis of MCR Breach Flood (June 20, 2011) -addresses major topics identified in the re-analysis document in response to items 10 - 11 (addresses MCR breach only), provided by PNL on August 17, 2011, 13 pp.
10b. Summary of Comments on Action13-4 (PNNL markup of June 20th submittal by H. Ahn, item 11c), provided by PNL on August 17, 2011, 108 pp.
10c. STP 2.4.5 (summary of the history of the probable maximum storm surge review) - provided by PNL on August 17, 2011, 3pp.
11a. Form 757 Attachment 1 -final2.doc (reasons for non-concurrence) - provided by H. Ahn, June 20, 2011, 4 pp.
11b. Enclosure 1 - (Re-analysis of MCR Breach Flood) provided by H. Ahn, Calculation Worksheet, June 20, 2011, 54 pp.
11.c. Enclosure 2: PNNL's Calculation Package/Worksheet, commented on by H. Ahn, June 20, 2011, 13 pp.

Note: Items 11a, 11b, and 11c are the NRC Form Section A materials submitted by H. Ahn

Summary of Recommendations: In the area of dam breach, recommended specific clarifications to the SER include the following actions for section 2.4.4:

- Clarification that the postulated storm surge will not cause significant erosion of the toe of the MCR embankment and thus would not lead to a breach in the MCR
- Clarification why overtopping within the MCR is not a cause for failure and breach
- Clarification why liquefaction within or in the vicinity of the MCR is not a cause for failure and breach

No further actions recommended in the area of storm surge and groundwater.

RR
Originally Submitted on December 20, 2011

CONTINUED IN SECTION D

SIGNATURE
Richard Raione

DATE
12-6-2012

SEE SECTION E FOR IMPLEMENTATION GUIDANCE

Attachment 1: Section B Summary and Recommendations by Richard Raione
December 20, 2011

As the Branch Chief of the Hydrologic Engineering Branch, I received a non-concurrence report on June 8, 2011, from one of my staff, Mr. Hosung Ahn. This non-concurrence is primarily concerned with whether or not the design basis flood (DBF) level of 40 feet mean sea level (msl) is adequate for the proposed South Texas Project (STP) Units 3 and 4 based upon two flood causal mechanisms. These two mechanisms include:

1) the design basis flood causal mechanism associated with a postulated main cooling reservoir (MCR) breach which is analyzed under NUREG 0800 SRP 2.4.4, and
2) a combination (cause/effect) scenario whereby a postulated hurricane storm surge could either exceed the MCR design basis flood level of 40 feet msl, or a postulated storm surge could theoretically cause an additive effect to the calculated DBF should a storm surge cause a breach in the MCR. Storm surge is analyzed under SRP 2.4.5.

Additionally, a third issue was brought up with the non-concurrence in the area of groundwater in terms of whether or not a groundwater departure was needed and how groundwater levels were reviewed.

Before the non-concurrence was filed, NRC staff and its contractor, Pacific Northwest National Laboratory (PNNL), worked closely together and met several times, to ensure that the applicant's analyses were technically correct and met regulatory guidance. A safety evaluation report without open items was finalized in May 2011 and staff agreed with STP's DBF level of 40 feet msl. Once the non-concurrence was filed in June 2011, input for the non-concurrence was provided to the PNNL contractor for their review and a response dated August 3, 2011, was received.

"Arms length" independent assessments by nationally recognized experts in the area of dam breach and storm surge were conducted. A total of six contracts were overseen by the NRC's Office of Regulatory Research and all reports were issued and finalized in the November – December 2011 timeframe by Dr. Donald Resio, Dr. Richard Leuttich and Dr. Jennifer Irish in the area of storm surge. For dam breach, Dr. Tony Wahl, Dr. Gregory Baecher and Dr. Richard Patev provided their respective independent reviews and assessments (additionally, they also assessed the potential impact of hurricane storm surge on the MCR).

Regarding dam breach, I am in agreement with the positions that all three contractors independently concluded. These summary statements include:

- *"Therefore, it is the opinion of the reviewer, that the breach analysis conducted by PNNL does follow a solid overall approach and a consistent methodology required for the breach analysis for the MCR embankment." Dr. Richard Patev, page 8.*

- *"In summary, the reviewer finds the MCR embankment breach analysis by the applicant, PNNL, and NRC staff as documented in the FSAR and SER to be technically adequate, conservative, and appropriate for establishing the design basis* *flood for safety related facilities associated with the expansion of the South Texas Project Electric Generating Station." Dr. Tony Wahl, page 20.*

- *"My conclusion is that PNNL's argument rests in large measure on the presumption that an overtopping failure is not possible. If that is accepted, then the issue of failure mode seems moot, as does the issue of low tailwater and broad downstream flows. Given that the normal maximum operating pool in the MCR is 49 feet and that the minimum crest elevation of the MCR embankment is 65.75 feet, the probability of overtopping due to rainfall appears negligible. ...the estimated maximum surge based on what appear to be conservative assumptions, and including a reasonable allowance for sea level rise, does not appear to pose a credible threat of causing an overtopping of the MCR, given the base elevation and height of the structure. Dr. Gregory Baecher, page 6.*

It should be noted that a probable maximum precipitation analysis was conducted for the MCR and results indicated that overtopping would not occur even with a PMP event. All three dam breach contractors were also in agreement regarding the potential for hurricane storm surge to cause a breach in the MCR as not credible. Additionally, Dr. Resio and Dr. Leuttich also expressed a storm surge impact to the MCR as not credible.

As a result of these independent reports in the area of dam breach, recommended specific clarifications to the SER include the following actions for section 2.4.4:

- Clarification that the postulated storm surge will not cause significant erosion of the toe of the MCR embankment and thus would not lead to a breach in the MCR

- Clarification why overtopping within the MCR is not a cause for failure and breach

- Clarification why liquefaction within or in the vicinity of the MCR is not a cause for failure and breach

On the topic of storm surge, two of three reviewers (Dr. Irish, Dr. Resio) stated that the applicant's ADCIRC model simulations were conservative and reasonable. Dr. Irish specifically states on page 10 of her report that:

"Overall, the computational methods employed to quantify hurricane flood elevation were reasonable and appropriately applied. Specific review of the ADCIRC analysis indicated that this model was appropriately configured and applied to simulate realistic hurricane flood elevations."

Dr. Resio on page 8 of his report provides a summary statement which reads "*In conclusion, I concur with the staff's decision on the licensing at this site*". Similar to Dr.

Leuttich and Dr. Irish, he does point out several findings that relate to ADCIRC inputs. Dr. Leuttich and Dr. Irish referred to Dr. Resio's expertise and modeling approach for resolution of their concerns.

All three reviewers expressed concerns about storm size and the storm's forward speed. However, Dr. Resio's report stated that even though the applicant used a non-state of the art storm parameter methodology (PMH/NWS-23), he concluded that the applicant's ADCIRC values were consistent with values he (Dr. Resio) would have obtained and that the applicant's values were overall conservative.

In consideration of the storm surge reports, it is recommended that no further action is needed.

Finally, regarding the topic of maximum groundwater level for the ABWR, the DCD Tier 1 limit is two feet below plant grade. The nonconcurrence states that the FSAR site characteristic is 28 ft msl for groundwater and this is correct. A surface water departure was implemented for the two proposed units in accordance to the DCD limit (a surface water departure is required for the ABWR if a DBF is shown to exist at a level equal to or higher than 1 foot below plant grade). For the proposed units at STP, this surface water departure equated to 33 ft msl. The NRO's Division of Engineering evaluated saturated conditions from 28 ft msl up to 33 ft msl, and of course evaluated the design basis flood impacts from 34 ft msl to 40 ft msl with no safety deficiencies noted. The nonconcurrence incorrectly puts the DCD term "maximum groundwater level" in the wrong context by failing to recognize that this particular requirement is valid only during a non-design basis flood event. Regarding this third topic, no further action is recommended.

Attachment 2: STP NCP Independent Review
Summary and Detailed Analyses
December 15, 2011

STP NCP Independent Review Summary

There were six independent reviewers of the non-concurrence process (NCP) documents: [1] Donald T. Resio[i], [2]Rick Luettich[ii], [3] Jennifer L. Irish[iii], [4] Tony L. Wahl[iv], [5] Gregory B. Baecher[v], and [6] Robert C. Patev[vi]. Reviewers [1],[2], and [3] focused on NCP comments related to storm surge and reviewers [4], [5] and [6] focused on NCP comments related to main cooling reservoir (MCR) breach analysis. The issues raised by the NCP petitioner regarding the positions put forth by the staff in the Safety Evaluation Report (SER) and by the applicant in the Final Safety Analysis Report (FSAR) are itemized in the following table along with a brief description of the independent review comments and any action to be taken by staff in consideration of the independent review comments. These issues and actions relate to SER Sections 2.4.4 and 2.4.5 and are indexed reflecting the appropriate SER section. Reviewers are referred to by numbers 1-6 as noted above.

Issue	Description	Review Comments	Action
2.4.4-NCP01	Inappropriate Empirical breach regression equation	[4] rejects the petitioner's argument. [5] agrees with staff on the appropriateness of the Froehlich equation but states that is has significant uncertainty that should be addressed. [5] further states that the petitioner's empirical fit does not seem to be an improvement of that of Froehlich which the staff used. [6] states that staff's selection and use of the Froehlich equation yielded reasonable breach width estimates.	No action
2.4.4-NCP02	Breach should be compared to historical levee breaches	[4] agrees with staff that an MCR breach would not be as severe as that of a levee for several reasons. [5] agrees with staff that historical levee failures noted in the NCP are not appropriate for comparison to the MCR. [5] states that petitioner's extreme conservatism is not supported by historical data and rejects the petitioner's conclusion that the staff's analysis is neither accurate nor conservative on this basis. [6] states that an MCR failure would more resemble that of a dam than a levee.	No action

2.4.4-NCP03	Inappropriate Mix-and-Match for breach analysis	[4] concludes that the approach used by staff has less uncertainty that other approaches and maintains an appropriate amount of conservatism.	No action
2.4.4-NCP04	Incorrect application of NWS-BREACH	[4] rejects the petitioner's argument.	Update 2.4.4 to state that [4] reviewed selection and use of BREACH and found it acceptable.
2.4.4-NCP05	Inappropriate value of Manning's n used in NWS-BREACH	[4] rejects the petitioner's argument.	Update 2.4.4 to state that [4] review parameter selection and found it to be very conservative.
2.4.4-NCP06	Failure to use Chow (1959) to adjust Manning's n	[4] rejects the petitioner's argument.	No action
2.4.4-NCP07	Failure to account for formation of a scour hole in breach analysis	[4] concludes that the effects of the scour hole are unproven as put forth by the petitioner. [5] agrees that the potential for scour hole exist and should have been address by staff but that geotechnical site considerations mitigate against formation of a scour hole. [6] recommends that the formation of scour hole is highly unlikely and does not support petitioner's argument that it should be included in staff's analysis and that a breach of the MCR embankment would not cause erosion below the foundation level.	Update 2.4.4 to state the reviews evaluation was that formation of scour hole was unlikely due to site soil characteristics.
2.4.4-NCP08	Failure to complete an appropriate sensitivity analysis of embankment width and tailwater cross-section	[4] concludes that while a sensitivity analysis would have been useful to include in staff's analysis, it is unlikely to change staff's FSAR and SER conclusions due to conservatism included in the analysis presented. [5] states that staff analysis should include an assessment of the validity of the assumed tailwater condition.	Update 2.4.4 to state that further sensitivity analysis would be useful based on reviews, [4] states it unlikely to change DBF.
2.4.4-NCP09	Inappropriate comparison of MCR to Martin Cooling Pond	[4] rejects the petitioner's argument. [5] states that comparison to Martin Cooling Pond has little value.	No action
2.4.4-NCP10	Failure to consider the storm surge as	[4] concludes that based on the duration of inundation, grass cover	Update 2.4.4 to reference 2.4.5 and

	mechanism leading to MCR embankment breach	and soil type significant erosion at the toe of the MCR embankment is unlikely cause an MCR breach as a result of storm surge.	then state that reviews support unlikely significant erosion due combination of unlikely to minimal level of inundation and site soil characteristics.
2.4.4-NCP11		[5] states staff should address uncertainty breach parameter estimates.	
2.4.4-NCP12		[5] states that staff's case for excluding overtopping is not clear. [6] states that overtopping of the MCR embankment is highly improbable.	Ensure that SER states the rationale for exclusion of overtopping as a potential cause of MCR embankment breach; restate if necessary.

References:

Baecher, GB. 2011. Independent Technical Review of The Main Cooling Reservoir Dam Breach. Memorandum to Joseph Kanney, USNRC dated 7 November 2011. Marked as draft.

Patev, RC. Undated. Independent Technical Review for the Main Cooling Reservoir Embankment at the South Texas Project Units 3 & 4. Risk Management Center, US Army Corps of Engineers.

Wahl, T. L. 2011. Review of Main Cooling Reservoir Embankment Breach Analysis: Expansion of South Texas Project Electric Generating Station. U.S. Department of the Interior, Bureau of Reclamation, Technical Service Center, Hydraulic Investigations and Laboratory Services Group. Denver, Co.

Irish, J. L. 2011. Independent Technical Review: South Texas Plant Units 3 and 4 Storm Surge Analysis. Draft Final Report dated 27 October 2011.

Luettich, R. 2011. External Review of non-concurring issues on the Safety Evaluation Report for the South Texas Project Combined License Application. Revised 21 November 2011.

Resio, D. T. 2011. Independent Review of South Texas Plant (STP) Units 3 & 4 Storm Surge Analysis. Taylor Engineering Research Institute, College of Computing, Engineering and Construction. University of North Florida. Jacksonville, Fl. Contact NRC-HQ-11-P-04-01.

Ahn, H. 2011. Enclosure 1: Re-Analysis of MCR Breach Flood. NRO/DSER/RHEB.

[i] Director, Taylor Engineering Research Institute, College of Computing, Engineering and Construction, University of North Floria, Jacksonville, Fl.
[ii] Director, Institute of Marine Sciences, University of North Carolina at Chapel Hill, NC.
[iii] Associate Professor, Civil Engineering, Virginia Tech, Blacksburg, Va.
[iv] Hydraulic Engineer, hydraulic Investigations and Laboratory Services Group, Bureau of Reclamation, Denver, Co.
[v] Professor, Civil and Environmental Engineering, University of Maryland, College Park, Md.
[vi] Senior Risk Advisor, Risk Management Center, US Army Corps of Engineers

The NRC non-concurrence process (NCP) document and Pacific Northwest National Laboratory's (PNNL) response to those were reviewed by an independent panel. The panel was also provided with the South Texas Project (STP) Units 3 and 4 Final Safety Analysis Report (FSAR) and the NRC staff's Safety Evaluation Report (SER). The panel consisted of:

Donald T. Resio, Director, Taylor Engineering Research Institute, College of Computing, Engineering and Construction, University of North Florida, Jacksonville (previously of USACE Coastal and Hydraulics Laboratory, Vicksburg) (Storm Surge)

Rick Luettich, Director, Institute of Marine Sciences, University of North Carolina at Chapel Hill (Storm Surge)

Jennifer L. Irish, Associate Professor, Civil Engineering, Virginia Tech., Blacksburg (Storm Surge)

Tony L. Wahl, Hydraulic Engineer, Hydraulic Investigations and Laboratory Services Group, Bureau of Reclamation, Denver (Dam Breach)

Gregory B. Baecher, Professor, Civil and Environmental Engineering, University of Maryland, College Park (Dam Breach)

Robert C. Patev, Senior Risk Advisor, Risk Management Center, US Army Corps of Engineers

Review reports from Resio, Leuttich, Irish, Wahl and Patev are final. NRC Research is waiting to hear some expanded justification for some of his conclusions from Baecher.

Summary of Dam Breach Review by the Panel

Tony Wahl

- On the NCP-raised issue that FSAR and SER inappropriately selected empirical breach parameter regression equations, the reviewer rejects the argument. The reviewer states that the Froehlich equation selected by the FSAR and the SER is an appropriate choice. The reviewer also states that the Macdonald and Langridge-Monopolis (MLM) equation, suggested as more appropriate in the NCP, actually results in greater uncertainty in its predictions than those from the Froehlich equation. The reviewer also states that using the peak discharge predicted from the Froehlich equation in combination with the time-to-failure predicted by the MLM equation resulted in a more conservative scenario in the FSAR because the time-to-failure predicted by the MLM equation is shorter.
- On the NCP-raised issue that the Main Cooling Reservoir (MCR) embankment failure should be compared with those of levees, the reviewer agrees with PNNL's opinion that levees typically experience larger breach width primarily because the duration of overtopping is long. The reviewer notes that the boundary condition at the upstream side of a levee is a live water body capable of maintaining a large driving head as the breach develops with the available quantity of

water far exceeding the breach flow. The reviewer points out that a dam typically has a finite-volume impoundment and therefore the upstream water level diminishes significantly as the breach forms and the impoundment drains. The reviewer states that the design and construction characteristics of the MCR are more like a dam than a levee and that the materials used in the MCR embankment are also much more like a typical dam than a typical levee.

- On the NCP-raise issue that the applicant used a mix-and-match approach, the reviewer states that "the selection of use of the mixed breach parameter equations effectively utilized those equations with demonstrably smaller uncertainty while maintaining appropriate conservatism."
- On the NCP-raised issue that the National Weather Service (NWS) BREACH computer model was incorrectly applied in the FSAR and the SER, the reviewer also rejects the argument. The reviewer states that the NWS-BREACH model is used to confirm the results from NWS FLDWAV model, which is the basis for the design basis flood analysis in the FSAR.
- On the NCP-raised issue that the Manning's n value for the breach channel was inappropriate, the reviewer rejects the argument. The reviewer explains the basis, history, and the role of Manning's n in NWS-BREACH. The reviewer illustrates what grain sizes would be associated with various Manning's n values and shows that Manning's n values exceeding 0.05 are implausible for the MCR breach analysis.
- On the NCP-suggested way to estimate Manning's n from the method described in Chow (1959), the reviewer rejects the suggested approach. The reviewer explains that accounting for effects from obstructions, vegetation, channel variability and other factors is inappropriate for a dam breach channel. The reviewer suggests that the Strickler equation as given in original NWS-BREACH documentation or other methods that estimate Manning's n based on materials forming breach channel be used.
- On the NCP-suggested effects of scour hole, the reviewer states that the NCP document does not provide enough details of how a scour hole was incorporated into the NWS-BREACH modeling. The reviewer points out that a physical mechanism describing how a scour hole would increase the breach flow is not provided in the NCP documents. The reviewer also states that the NCP may have lowered the tailwater section downstream from the embankment which may have resulted in a larger head to drive the breach flow or reduced the chance that backwater effects would retard the breach flow. The reviewer notes that this (speculated) approach in the NCP would ignore that the tailwater channel further downstream would cause its own backwater effect and may submerge the (artificially-created) scour hole. The reviewer concludes that the effects of a scour hole are unproven.
- One the NCP-raise issue of sensitivity to embankment width and tailwater cross-section, the reviewer concludes that with a reasonable Manning's n, the sensitivity of NWS-BREACH predictions to embankment length and tailwater channel size are insufficient to produce peak outflows that would change FSAR and SER conclusions. The reviewer does state that it would have been advisable for PNNL to perform sensitivity analyses to embankment length and tailwater cross-section with a Manning'n n of 0.075. However, the reviewer concludes that "without also increasing the tailwater section, they would have still concluded that an embankment length of 1000 ft did not constrain the resulting breach width." Also, the reviewer

reiterates: "... that values of n greater than 0.050 are inappropriately large; with n=0.050 or lower, the embankment length of 1000 ft used by PNNL would have been adequate to avoid constraining the breach width, even with a larger tailwater cross section."

- On the NCP-raised issue that comparison of MCR breach event to the Martin Cooling Pond case is inappropriate, the reviewer rejects the argument. The reviewer independently confirmed the properties of the Martin Cooling Pond and its failure. The reviewer concluded that the comparison made by PNNL is appropriate and confirmed that the results of other analyses are reasonable.
- On the NCP-raised issue of the potential for hurricane storm surge to cause a failure of the MCR embankment, the reviewer states that the failure mode is not credible because the presumed loading primarily attacks the toe of the embankment for a short duration and the grass cover is likely to prevent erosion into the embankment during the loading time period. The reviewer also states that the embankment soils themselves are at least moderately resistant.

Gregory Baecher

- The reviewer states that "the NRC non-concurrence (#4) stretches its arguments in favor of the most extreme condition beyond the support of historical data on dam breaches and beyond what appears to be the actual geometry and loading patterns of the MCR. I do not think that the arguments of the NRC non-concurrence by themselves fully support the conclusion that the PNNL analysis is "neither accurate nor conservative."
- The reviewer states that "On the other hand, the PNNL analysis and response (#7) to the NRC non-concurrence seems to overstate its case, ignore significant statistical uncertainties, over-simplify potential loading events, and make a poor case for ignoring multiple failure modes."
- The reviewer also states that "A major limitation of both the PNNL analysis and the USNRC non-concurrence is that they fail to adequately address significant epistemic uncertainties in the modeling."
- The reviewer states that "at the same time the PNNL analysis seems inadequate for other reasons. Whether these other reasons are of sufficient importance and whether they have sufficient impact on the uncertainties attending the breach flood prediction to negate the conclusions of the PNNL analysis is outside the scope of this review, which precludes any re-analysis."
- The reviewer states that the NCP and PNNL analysis appear to hinge on (a) "whether the MCR 'fails in the pattern of a dam or a levee'" and (b) "Thus the character of the tailwater downstream of the breach, and thus the outflow hydrograph." The reviewer states that the two points above then depend on the failure modes considered and the internal design of the MCR embankment. The reviewer states that the prediction of outflow and downstream flooding depend on the "choice of the best forecasting model of breach discharge" and "whether a scour hole forms at the toe of the breach."
- The reviewer states that the NCP documents and the PNNL analysis and response do not identify the degree of uncertainty in predictions of breach discharge "attending statistical regression analysis."

- The reviewer states that the NCP argument that "the MCR will behave as levees do when they fail" appears not well-supported.
- The reviewer states that the NCP "uses a statistical correlation of outflow discharge to reservoir characteristics" (presumably the MLM equation and an equation refit to reservoir and dam breach data by the NCP author) that "does not seem as good as the Froehlich correlation that the PNNL analysis is based on."
- The reviewer states that PNNL analysis has four flaws: (a) ignores all but piping failure modes, (b) fails to account for large uncertainty in the Froehlich correlation, (c) dismisses the contention that tailwater conditions may be much different than assumed, and (d) dismisses the possibility of a scour hole.
- On whether the MCR fails as a dam or a levee:
    - The reviewer states that (a) PNNL's approach is based on piping failure, which leads to a narrow outflow channel, higher tailwater, rapid transition to subcritical flow, and lessened discharge and (b) NCP argument is that an overtopping failure or an unraveling failure such as in levees would lead to a broad outflow sheet, lower tailwater, slow transition to subcritical flow, and heightened discharge. The reviewer states that each of these assertions seems correct, given their respective presumptions.
- On failure modes and internal design of the MCR embankment
    - The reviewer states that PNNL mentioned three failure modes in the "report for the MCR"—seismically-induced soil liquefaction, overtopping due to high reservoir inflows, and internal erosion leading to piping. The reviewer mentions that historically, 40% of earth dam failures are due to piping, 40% due to overtopping or spillway inadequacy, and 20% due to slides or miscellaneous factors. The reviewer mentions that a dam failure by high reservoir inflows during a hurricane would seem more likely to be caused by overtopping rather than piping.
    - The reviewer states that "PNNL argues that the Bureau of Reclamation's Dam Safety Office (USBRGDSO) database supports the claim that the MCR embankment will fail in a narrow-deep breach." The reviewer states that this seems an inappropriate conclusion.
    - The reviewer states that the Martin Cooling Pond failure "does not shed light on the likelihood of an extended breach due to overtopping." The reviewer also states that many other historical examples (presumably in the USDSO database) "similarly seem to differ enough from the MCR embankment as not to shed much light on the current site."
    - The reviewer states that PNNL argues the embankment will not overtop because of insufficient inflow to the MCR. The reviewer states that "if the hydrology-hurricane review substantiates the contention that the overtopping probability is diminishingly small, then this failure mode could be dispensed with, and the presumption of only deep-narrow breaching due to piping would be supported." (The UFSAR for Units 1 and 2, the FSAR for Units 3 and 4, and the SER all document that overtopping of the MCR embankment is implausible.)

- The reviewer states that comparing the MCR embankment to the California Delta levees or flood levees on the Danube is inappropriate.
- The reviewer concludes that "PNNL's argument rests in large measure on the presumption that an overtopping failure is not possible." Further, "of that is accepted, then the issue of failure mode seems moot, as does the issue of low tailwater and broad downstream flows. If that presumption is not accepted, then the more severe inundation of the non-concurrence have at least some credibility. However, the historical evidence for broad breaching in reservoir impoundments is sparse."

- On choice of the best forecasting model of breach discharge (MLM vs. Froehlich methods):
  - The reviewer notes that many regression models have been published and most of the models have been calibrated to more or less the same (historical) cases.
  - The reviewer states that the best-fitting model today appears to be the due to Froehlich, which is a bivariate model. The reviewer also states that the many competing model either use only one independent variable or combine volume and depth (of the reservoir) in a single factor. The reviewer states that this leads to lower R^2 statistics and larger predictive errors when compared to Froehlich's bivariate model.
  - The reviewer stated that PNNL's choice of Froehlich's model seems appropriate. The reviewer also states that the NCP uses MLM model that provides a poorer fit to historical data compared to Froehlich;s model and thus "seems an inferior choice by a substantial margin."
- On development of breach scour hole:
  - *The reviewer agrees with the NCP statement that development of scour holes at the site* of an embankment breach, especially for levees on natural ground, is common. The reviewer also states that it seems likely that formation of a scour hole would increase the breach size and lead to higher outflow discharges. (The reviewer does not say why.) The reviewer states that PNNL should have investigated this possibility.
  - The reviewer also states that the geotechnical conditions at the MCR embankment site, while no guarantee against the formation of a significant scour hole, do mitigate against scour.
- On uncertainty in predictions:
  - The reviewer states that even using the Froehlich regression, the uncertainty in peak breach discharge is large, "on the order of a ±two-standarad-deviation range from ½ to 2." The reviewer states that using the best estimate is not conservative for the purposes of risk assessment. The reviewer states "an annual exceedance function of probability is called for, even if the non-concurring review did not identify such."

Robert Patev

- The reviewer states that he conducted a review focused on the geotechnical aspects of a failure of the northern MCR embankment.
- The reviewer states that the design, materials, construction methods, and seepage control in the MCR embankment are critical factors that affect failure mode.

- The reviewer notes that the MCR embankment consists of low-permeability material which makes seepage failure highly unlikely.
- The reviewer states that the MCR embankment foundation material consists of two different high-strength clays that will reduce the chances of foundation erosion.
- The reviewer notes that the seepage control system for the MCR is well-designed.
- The reviewer concludes that an overtopping failure of the MCR embankment, including a potential overtopping from storm surge, is highly improbable.
- The reviewer concludes that erosion of the soil-cement barrier leading to a breach is also highly improbable.
- The reviewer concludes that seepage and piping through and under the MCR embankment are credible and likely failure modes.
- The reviewer recommends that a scour hole in the breach analysis be not included because the of the two clay strata that are moderately to very stiff and therefore, in the reviewer's opinion, would make erosion of the foundation highly unlikely. The reviewer cites similar conditions that occurred during Hurricane Katrina where the stiff clays in the foundation did not significantly erode away.
- The reviewer states that the loss of containment and breach of the MCR embankment would resemble more that of a failed dam than that of a failed levee section because "the MCR reservoir would breach and lose its containment very quickly." The reviewer states that riverine levee breaches are different because the breaches occur over a much longer period of time and there is (virtually) infinite water available during the breach formation and progression.
- The reviewer states that the majority of the material in the MCR embankment is compacted silty clay which is a medium to low erodibility material. The reviewer states that the sand material would be removed rapidly following the breach but would not expedite the seepage path because the pipe would need to intersect another clay layer before it can exit the embankment. The reviewer concludes that the breach would evolve slowly and then erode most of the embankment material down close to but above the foundation level.
- The reviewer states that Froehlich approach would lead to a reasonable estimate of breach width. The reviewer notes that because the Froehlich method gives the least amount of dispersion in its prediction, a breach width between 400 and 600 ft would be a reasonable estimate for the MCR embankment.
- The reviewer notes that the MLM equation gives a reasonable prediction of time to peak outflow but the Froehlich approach seems too conservative.

Summary of Storm Surge Review by the Panel

Don Resio

- The reviewer listed five components:
    - An overview of the original work and concerns raised in the non-concurrence opinion
    - A review of the storm surge parameter selection and application to modeling

- A review of wind and surge modeling element involves in the estimation of the Probable Maximum Storm Surge (PMSS) at the STP site
    - A review of the adequacy of the storm surge analysis conducted by the applicant and PNNL with respect to specific storm surge levels affecting the dam breach analysis
    - A summary of the findings and presentation of final conclusions
- Overview of original work and issues raised in the non-concurrence:
    - The reviewer states that some of the applicant's methods are not totally consistent with older standards or those used in recent studies by Federal agencies. (Presumably the reviewer means storm surge modeling approaches by "standards.")
    - The reviewer states that it would have been better for the applicant to select one of the methods used in a large study because such models have already been carefully calibrated and validated. The reviewer notes that lacking validation, he assumes that the applicant's methods did not introduce large bias compared to previous methods and notes that he had to rely on his knowledge of past modeling results to ascertain the adequacy of FSAR and SER-reported results.
    - The reviewer states that the knowledge regarding hurricanes affecting the Gulf coast of the US has advanced significantly since the National Weather Service (NWS) Report 23 (NWS 23) was published. The reviewer notes that much of the new knowledge is not consistent with the concepts used in NWS 23.
    - The reviewer states that results from SURGE model combined with HEC-RAS are outdated and should be considered irrelevant for establishing a credible surge level at the STP site. (PNNL and the NRC staff arrived at the same conclusion in the SER.)
    - The reviewer states that there is a need to understand and quantify all uncertainties in critical decision-making. The reviewer notes that the FSAR and SER approach for estimation of the probable maximum storm surge (PMSS) is deterministic and assumes negligible uncertainty. The reviewer further states that uncertainty in the probabilistic equivalent of the PMSS, which is hypothesized to have an annual exceedance probability of no more than 1 in 1,000,000, is not negligible.
- Review of the PMH parameter selection:
    - The reviewer states that the applicant has followed NWS 23 reasonably. However, the reviewer states, there are six parameters used in modern (presumably post-NWS 23) storm surge modeling: storm intensity, storm size, storm forward speed, storm track angle, landfall location, and peakedness of the wind speed distribution within the storm. The reviewer notes that the FSAR and SER tacitly assume the Holland B parameter to have a value of 1.
    - The reviewer states that the NWS Sea, Lake, and Overland Surges from Hurricanes (SLOSH) model is not believed to be a proper model to use in modern applications in which flooding far inland is being investigated. The reviewer approves the use of the Advanced Circulation (ADCIRC) model because it has been calibrated and validated for both coastal and inland sites in very large studies conducted by US Army Corps of Engineers (USACE) and the Federal Emergency Management Agency (FEMA).

- The reviewer states that the applicant neglected the variations in storm intensity and size as they approach the coast. The reviewer states that such variations are incorporated in virtually all modern studies of hurricane characteristics including those conducted by USACE and FEMA. The reviewer notes, however, that "this neglect is somewhat fortuitous," because it adds a degree of conservatism which compensates for some under-conservative parameter choices. (Note that the FSAR or the SER did not unknowingly neglect the weakening of storms as they approach the coast—this assumption was expressly used to add conservatism to PMSS.)
- The reviewer stated the pressure differential in the FSAR and SER is 133 mb (1020 mb peripheral pressure minus 887 mb central pressure for the PMH). The reviewer states that a value of 887 mb for central pressure is not much lower than relatively short historical observation, about 60 years. The reviewer states that Hurricane Rita was estimated to have a lowest central pressure inside the Gulf of Mexico of 895 mb and Hurricane Wilma, just before it reached the Yucatan peninsula, had an estimated central pressure of 882 mb. The reviewer feels that it may be prudent to take the lowest central pressure for hurricanes in the Gulf of Mexico as 880 mb because 887 mb is not very conservative by modern standards. The reviewer states that a more typical value of peripheral pressure in the Gulf of Mexico may be taken as 1015 mb rather than the FSAR and SER value of 1020 mb. So, the reviewer states that "the total impact on the pressure differential of changing these two factors would be about 2 mb which is probably relatively negligible."
- The reviewer states that a more significant problem is the value of storm size used in the FSAR and the SER. The reviewer notes that the SER shows that the largest storm simulated is only 20.9 nm, which is only slightly larger than the median value for storms along the Texas coast. The reviewer states that this value for storm size is not conservative. The reviewer states that Irish et al. (2008) have shown that storm size is a critical hurricane parameter and strongly influences storm surge levels.
- The reviewer states that the PMH values used for track angle and forward storm speed appear reasonable.
- The reviewer states that clearly, larger storms than the PMH storm size used in the FSAR and the SER have occurred (Hurricane Carla had a radius to maximum winds [RMW] of about 35 nm). The reviewer alse states that increasing the storm size will produce an increase in the storm surge levels at the STP site.
- The reviewer states that the neglect of the Holland B parameter would also create a tendency to produce under-conservative surges. The reviewer states, however, that because the average value of the Holland B parameter in the Gulf of Mexico is about 1.3, the effect of neglecting this parameter is likely to be relatively minor.
- The reviewer states that as a storm approaches the coast, its intensity diminishes and the size increases, affecting the storm surge in opposite directions—a decrease in storm intensity will naturally diminish the surge while an increase in storm size will naturally increase the surge. The reviewer states that the relative effects of the two factors is addressed by Irish and Resio (2010), where it is shown that storm surges tend to

increase asymptotically with storm size but do not appear to have a similar asymptotic limit with storm intensity. The reviewer estimates that an FSAR-postulated PMH which undergoes near-coast evolution (presumably increase in storm size and decrease in storm intensity), will have its central pressure increase by 14.9 mb. The reviewer also states that storm surges tend to vary linearly with storm size up to about 25 nm and then asymptotically increase until a limiting value is reached at about 30-35 nm. The reviewer states, based on the work by Irish et al. (2008) that the over-prediction in storm surge from neglecting storm intensity decay would be about 8-12% while the under-prediction associated with the selection of a smaller storm size (35-20.9=14.1 nm) would be 7-10%. The reviewer concludes that because the compensating effects are close in magnitude, the FSAR results will likely be consistent with values obtained via a more modern approach.

- Review of wind and surge modeling elements used for estimating the PMSS at the STP site:
    - *The reviewer states that SURGE-HEC-RAS simulation to estimate storm surge is inappropriate.*
    - The reviewer states the storm surge analysis using the SLOSH model is an improvement but this application is over-simplified compared to modern storm surge modeling efforts. The reviewer states that although the SLOSH model has been shown to be relatively accurate (Jarvinen and Lawrence, 1985), the accuracy is extremely dependent on the value of the offshore boundary conditions, which is allowed to vary somewhat as a fitting function in typical SLOSH simulations. The reviewer states that without "appropriate" offshore boundary conditions, the SLOSH model underpredicts the surge because of its limited offshore spatial domain. (Note that PNNL used a 10%-exceedance tide plus initial rise plus long-term sea-level rise to specify initial water level across the whole SLOSH domain. The reviewer does not say if this assumption is conservative or not. The applicant used the same approach as the staff/PNNL.)
    - The reviewer states that the lack of inland extent and (spatial) resolution in the SLOSH simulation domain are expected to lead to spurious over and under-predictions throughout the inland area and therefore the reviewer concluded it is difficult to appreciate the applicability of SLOSH simulations for the STP site.
    - The reviewer states that the staff's more conservative estimates of the "maximum PMH" are probably the most reasonable estimates among the entire set of estimated, if involved uncertainties are included. The reviewer also states that the applicant's estimate should be considered consistent with estimated which might be obtained via careful deterministic methods. (Presumably he means SLOSH simulations performed by staff/PNNL and ADCIRC simulations performed by the applicant.)
- Review storm surge analysis with respect to specific storm surge level affecting the dam breach analysis:
    - The reviewer concurs with the staff's conclusion that the effects of wave set-up and run-up on the MCR embankment based on a deterministic analysis of the PMSS.
- Summary of findings and final conclusions:

- The reviewer states that SURGE and HEC-RAS tools are inappropriate for storm surge analysis.
- The reviewer states that applicant of SLOSH is an improvement over SURGE. The reviewer stated that the lack of careful quantitative methods to estimate water levels at offshore and lateral boundaries of the SLOSH grid makes the application of the SLOSH model to PMH very questionable. The reviewer states that lack of inland extent and resolution of typical SLOSH grids makes for a poor choice for determination of PMH flooding.
- The reviewer states that ADCIRC runs should have been conducted with wind fields for which the ADCIRC model has been calibrated in recent large studies. The reviewer states that his belief is that the overall effect of this choice on model predictions should be fairly small.
- The reviewer states that use of a radius to maximum wind speed of only 20.9 nm is too small by modern standards because larger storms have been observed in the Gulf of Mexico in the last 60 years (Hurricanes Carla and Ike). The reviewer notes that larger values for the radius to maximum winds in a PMH would produce a net increase in estimated storm surge levels.
- The reviewer states that the applicant did not include the effects of storm intensity decay as the PMH approaches the coast. The reviewer states that storm intensity decay would result in a net decrease in the estimated storm surge.
- The reviewer states that based on works by Irish et al. (2008), Resio et al. (2009), and Irish and Resio (2010), the net effect of all of the deviations in the specification of storm size and pressure differential is expected to be fairly small because two non-negligible terms of opposite sign end up with about the same magnitude. The reviewer noted that this is somewhat fortuitous and cannot be depended on for future licensing decisions.
- The reviewer concurs with the staff's decisions for licensing at the STP site, but limits his concurrence to the deterministic framework of the PMH/PMSS estimation. The reviewer recommends that some level of uncertainty analysis be included in the design levels.

Rick Leuttich

- The reviewer states that the non-concurrence conclusion reached by the NCP author is justified. The reviewer states that while he believes that ADCIR is the appropriate model to use to evaluate the PMH, he does not believe that the applicant applied the model in a conservative manner because
  (a) The PMH radius to maximum winds is underestimated in NWS 23. The calculated values are less than several recent storms that have made landfall in the Gulf of Mexico.
  (b) The applicant cited historical storms that were used to validate ADCIRC along the Texas coast, but none of the material was published or provided to the reviewer. The reviewer states, therefore, that it is not possible for him to assess the ADCIRC skill or overall uncertainty in the vicinity of the STP site.

(c) The reviewer does not believe that ADCIRC runs using a moderately-sized, fast moving storm that ramps up from category 1 to category 5 over the 12 hours prior to landfall will lead to an accurate representation of the PMSS at the STP site. The reviewer believes that this conclusion is based on flawed SLOSH runs.

(d) The reviewer states that the several evaluations of the PMSS using SLOSH (SLOSH MOMs analysis, NRC staff runs, applicant runs, and Vickery [2009]) are unreliable and should not be used.

(e) The reviewer states that the Resio (2009) study appears to be more thorough. The reviewer states that while the Resio (2009) PowerPoint presentation was short on details, its maximum ADCIRC surge value of approximately 30 ft presumably represents the PMSS. The reviewer notes that since the Resio (2009) study, ADCIRC was substantially recalibrated for Hurricane Ike and it is not clear how this recalibration would affect Resio (2009) results. The reviewer recommends repeating one or two of the runs that resulted in the greatest surge levels near the STP site using the recalibrated ADCIRC. The reviewer states that if the results of the reruns are comparable to Resio (2009) results, they likely represent the PMSS at the site and if the recalibrated ADCIRC results are greater than Resio (2009) results, a larger set of runs should be repeated to identify the PMSS.

(f) The reviewer states that the adequacy of the applicant's analysis for the potential for breach of the MCR embankment is a moot point because only the NRC SLOSH results indicate surge would reach the area of concern and the reviewer does not believe these are credible results. The reviewer states that if additional ADCIRC runs indicate inundation of the northern embankment, the velocity field generated by ADCIRC and wave field generated by SWAN or STWAVE should be used to assess potential erosive forces.

- On probable maximum winds and associated meteorological parameters:
  - The reviewer notes that given the landfall conditions of the strongest of the recent storms in the Gulf of Mexico (Hurricane Katrina—Category 3, RMW=30 mi; Hurricane Rita—Category 3, RMW=23 mi; Hurricane Gustav—Category 2, RMW=27 mi; Hurricane Ike—Category 2, RMW=35 mi), the PMH RMW (24 mi) in NWS 23 is underestimated. The reviewer states that it is critical that upper end RMW values be considered when determining the PMSS and it appears that only Resio (2009) includes larger value of RMW at landfall. The reviewer recommends that at a minimum, the PMSS due to a Category 3-4 storm with large RMW should be compared with that already computed for a Category 5 storm with smaller RMW to determine the most conservative conditions. (Note that Resio made a comparison of the effects of increased storm size and reduced intensity in his review.)
  - The reviewer states that increasing evidence suggests that storms will become stronger as the climate warms, although it is not clear how the entire suite of storm parameters, including RMW will be affected. The reviewer states that the applicant's conclusion that PMH parameters from NWS 23 account for any increase in hurricane strength due to future climate variability is not well justified. The reviewer recommends that at a

minimum, the most conservative PMSS computed using present-day PMH parameters should be determined, e.g., using the larger storm size and sensitivity to increased storm strength and size by 10-20% should be evaluated.

- On surge and seiche water levels:
  - The reviewer states that while the SURGE model may provide a quick sanity check on surge calculations in open coastal areas, it lacks important physical processes that contribute significantly to surge and therefore does not represent the current state of practice. The reviewer also states that coupling with HEC-RAS to compute inundation and water levels is not reliable. The reviewer recommends that SURGE/HEC-RAS model combination be discounted for the current study.
  - The reviewer states that while the extrapolation of SLOSH MOMs may provide a reasonable estimate of the corresponding SLOSH MOMs for the higher wind speeds of the PMH, it does not address underestimates of the surge due to small PMH RMW or fundamental errors in SLOSH due to poor grid resolution, inadequate domain size, or specific parameter values. The reviewer recommends that extrapolate SLOSH MOMs should be discounted for the current study.
  - The reviewer states that the NRC staff SLOSH runs confirmed that the greatest surge was generated by the largest RMW. The reviewer notes that because the PMH RMW may be underestimated, the PMSS may also be underestimated. The reviewer also states that a recent evaluation of the SLOSH model for Hurricane Ike has identified that the small SLOSH domain size and possibly frictional parameterization and lack of integrated wave forcing may result in underestimation of the Ike storm surge by 25 to 33% (about 1 to 2 m) compared to observations and ADCIRC model results. The reviewer postulated that this underprediction may result from the SLOSH model's inability to capture the 2-3 m geostophic setup the results from shore-parallel winds that occurred the day preceding Ike's landfall and raised the coastal and inland water level prior to the arrival of the main surge-generating winds. The reviewer notes that RG 1.59 suggests an initial rise in water level of 0.75 m, which is far below the Ike setup, particularly if steric effect is included. The reviewer stated that SLOSH's inability to represent the geostophic setup may have caused the NRC staff to conclude that a fast approaching storm would cause the highest surge. The reviewer states that in his opinion, a slower moving, large storm provides time for the generation of the geostophic setup that SLOSH misses altogether. The reviewer notes that his opinion is consistent with Resio (2009) study which showed that greater surge is caused by storms with slower forward speeds.
  - The reviewer stated that it appears that the ADCIR setup of the model grid and spatially-varying model parameters specification was done in a reasonable manner. The reviewer notes that he was not provided with validation details of ADCIRC applied to the STP site. The reviewer notes that the applicant replaced the Holland radial wind profile with the SLOSH radial wind profile in the ADCIR code. The reviewer notes that this change does not necessarily mean that ADCIRC and SLOSH both generate the same wind velocities; e.g., the reviewer states that ADCIRC converts its winds from an assumed 1 min

averaging time to a 10 minute averaging interval but the reviewer is unaware of a similar SLOSH approach. The reviewer also points out other subtle differences between ADCIRC and SLOSH and concludes that ADCIRC wind forcing may have been less than SLOSH. The reviewer points out that recent modeling for Hurricane Ike shows that ADCIRC generates larger surge compared to SLOSH. The reviewer recommends that explicit validation of the version of ADCIR used in the current study should be provided in the vicinity of the STP site. The reviewer states that additional ADCIRC model runs may be needed to evaluate the PMSS at the STP site. The reviewer also recommends that one or two of the Resio (2009) runs be re-performed with the re-calibrated ADCIRC and compared with Resio (2009) results. The reviewer states that if the results from the re-calibrated ADCIRC are comparable to Resio (2009), it is highly likely that these accurately represent the PMSS at the site.

- On wave action:
  - The reviewer states that the NRC staff's approach for wind wave estimation appears reasonable. The reviewer states that ADCIRC coupled to SWAN or STWAVE will provide estimates of wave properties throughout the region of inundation. The reviewer recommends making use of the wave properties directly from ADCIRC coupled with SWAN or STWAVE if additional model runs indicate inundation.
- On flooding protection requirements:
  - The reviewer summarizes the staff's review of the possibility of MCR embankment failure due to PMSS sloshing action. The reviewer states that the issue seems rather academic because only the NRC SLOSH results indicate surge would reach the area of concern. The reviewer does not believe that NRC staff's SLOSH results are credible and therefore an erosion analysis of the MCR's embankment is a moot point. The reviewer recommends making use of the wave properties directly from ADCIRC coupled with SWAN or STWAVE if additional model runs indicate inundation to evaluate potential erosive forces.

Jennifer Irish

- On FSAR:
  - PMH
    - The reviewer states that the applicant's upper value of RMW of 21 nm (39 km) is not representative of the upper limit for RMW for landfalls along the US Gulf of Mexico coastline. The reviewer states that since 1940, there have been 10 hurricanes with RMW exceeding the applicant-selected value; 1961 Hurricane Carla (56 km), 1965 Hurricane Betsy (74 km), and 2008 Hurricane Ike (estimated 50 km) are notable.
    - The reviewer states that recent hurricanes—Katrina, Rita, and Ike revealed the important role of storm size in surge generation over shallow shelves (Irish et al., 2008). The reviewer notes that all else being equal, a large radius storm will produce larger surge. The reviewer states that it is the combination of high

winds and size that governs surge generation and it does not necessarily follow that the storm with the strongest maximum winds will generate the largest surge (Irish and Resio, 2010). The reviewer also notes that there is a correlation between storm size and storm central pressure, where more intense storms are often of smaller size.

    - The reviewer states that the central pressure deficit used was 133 mb, which is conservative or extreme because it is about 30% larger than the largest central pressure deficit appearing in the US historical records for landfalls along the Gulf of Mexico since the 1940s. The reviewer states that the applicant's value for the central pressure deficit is close to the current-day minimum probable intensity for the Gulf of Mexico, on the order of 133 mb (Irish et al., 2008; Tonkin et al., 2000). (Did she mean maximum probable intensity?)
    - The reviewer notes that the applicant's not including storm intensity decay as the hurricane approaches the coast, adds conservatism. The reviewer also states that the applicant does not consider potential future hurricane intensification and its implications.
    - The reviewer states that in the vicinity of Matagorda Bay, she and her colleagues have found that maximum surge occurs when the storm is located on average 0.87 times the "pressure radius to the left of the location of interest" (Irish et al., 2009). The reviewer notes that this finding is close to the 1 RMW assumed by the applicant.
- Storm surge analysis approach
    - The reviewer notes that SURGE is an over-simplified model for hurricane surge estimation. The reviewer notes that the applicant concluded using SURGE model results that highest surges occur for large, fast-moving storms.
    - The reviewer states that the SLOSH model is in regular use by National Oceanic and Atmospheric Administration (NOAA) for hurricane surge forecasting and is an improvement over the SURGE model. The reviewer notes that a main drawback of the SLOSH model is that it does not use fine spatial resolution in order to increase computational speed; the coarse spatial resolution results in loss of accuracy.
    - The reviewer states that the applicant's extrapolation approach, that uses SLOSH MOMs, appears reasonable because wind speed is related to the square root of the central pressure deficit. The reviewer notes however, that a very limited number of data points were used and therefore a third-degree polynomial may not be justified. The reviewer states that the applicant again concluded, based on this analysis, that large, fast-moving storms generated highest surges.
    - The reviewer states that the ADCIRC model is widely accepted as an accurate model for hurricane surge estimation. The reviewer states that ADCIRC, applied at high resolution, represents the state-of-the-art for hurricane surge simulation. The reviewer notes that some of the conservatism considered in

applying both SURGE and SLOSH was not used in the applicant's ADCIRC simulations—specifically, a variable bottom friction, vegetation wind masking, and small-scale topographic barriers were used. The reviewer notes that including these three factors would lead to a more realistic representation of current-day conditions and most likely lead to lower flood elevation estimate at many locations. The reviewer states that the combination of the three factors along with significant differences in spatial resolution are likely factors that led to significantly lower flood estimates from ADCIRC.

- Analysis of other coastal flood level contributions
  - The reviewer states the USACE methods used to quantify wave height and runup at the MCR embankment are standard methods for estimation of simple wind-wave height and runup. The reviewer concludes that the addition of runup to the surge estimate from SURGE and SLOSH models is a conservative approach for SLOSH because this model includes an empirical correction for wave setup and therefore, adding runup in this way effectively double counts wave setup.
  - The reviewer stated that the applicant considered additional elevation due to astronomical tides and wave setup. The reviewer states that considering astronomical tides as additive is conservative.
  - The reviewer notes that the applicant considered possible future changes due to sea-level rise only assuming that the historical sea-level rise rate would continue in the future and no consideration was given to potential future sea-level rise acceleration.
- Analysis specific to a surge-induced MCR breach
  - The reviewer states that the applicant's conclusion that the risk of an MCR embankment breach caused by the PMSS is low is reasonable.
  - The reviewer states that the applicant's assumption that the dominant wind direction would be pointing away from the site (I think she means the northern MCR embankment) is reasonable if the high surge coincides with landfall.
  - The reviewer states that the three methods used by the applicant to estimate currents appear to be reasonable engineering approximations.
  - The reviewer states that the applicant's wind wave methods are standard practice and appear reasonable.
- Validity of Surge Analysis Results
  - The reviewer states that in general the applicant's surge modeling approach appears thorough and reasonably conservative.
  - The reviewer states as her primary concern the omission of large-radius hurricanes. The reviewer notes that very large hurricanes typically do not support very low central pressures; however, the reviewer states it would be prudent to consider the effects of intense, large hurricanes.
  - The reviewer states that a secondary concern is the non-conservative setup for ADCIRC, specifically the use of bottom characteristics. The reviewer states that

it would be useful to demonstrate that ADCIRC would predict flood levels at or below those estimated by SURGE and SLOSH models when non-conservative bottom friction, wind masking, and small-scale feature assumptions are applied. (I think she meant to say "conservative bottom friction, wind masking, and small-scale feature assumptions are applied.")

        - The reviewer notes two additional concerns—details of wave runup calculations and implementation with ADCIRC flood estimates and potential implications of future climate change and possible acceleration of sea-level rise—that are missing from the applicant's analyses in the FSAR.
    - Suitability of Surge Results for MCR Breach Analysis
        - The reviewer notes two additional concerns: (a) it would be valuable to validate the simplified methods used to estimate current speed from the SLOSH simulations by applying the methods to ADCIRC flood elevations and comparing with ADCIRC-simulated current speeds and (b) it would be prudent to review ADCIRC-simulated surge time series at the MCR embankment to validate the conclusions regarding high surge and wave directions.
- On SER:
    - PMH
        - The reviewer notes that the SER does note that other contemporary work suggests use of larger hurricane sizes.
    - Storm Surge Analysis Approach
        - The reviewer notes that PNNL did not perform confirmatory calculations using ADCIRC as it did for SLOSH. The reviewer states that PNNL does not mention non-conservative assumptions related to current-day friction, wind masking, and small-scale features. The reviewer states that a concern remains that this scenario may not be maintained in the future due to land development or damages incurred during storms.
    - Analysis of Other Coastal Flood Level Contributions
        - The reviewer notes that PNNL states wave runup is not included in ADCIRC output and therefore PNNL added the applicant estimate of runup to the ADCIRC water level. The reviewer states that this approach is conservative because it double-counts wave setup.
    - Validity of Surge Analysis Results
        - The reviewer notes that a failing of the SER review appears to be in not exploring hazards posed by large hurricanes.
    - Suitability of Surge Results for MCR Breach Analysis
        - The reviewer mentions no additional concerns.
- On NCP Issues 1 and 2:
    - The reviewer states that PNNL, in its response to worst case PMH event, makes no specific mention of their findings regarding hurricane size.
    - The reviewer states that the computational methods employed in arriving at the 39.9 ft flood estimate appear sound and given the conservatism of this analysis the additional

0.1 ft margin appears suitable. The reviewer expresses concern that large hurricanes, that appear in the historical record and have occurred recently (Katina 2005, Rita 2005, and Ike 2008) have not been considered.

- The reviewer mentions that although PNNL did not perform independent ADCIRC confirmatory runs, it made a thorough study of the information provided by the applicant and supplemented with additional literature and information available on the use of ADCIRC. The reviewer concludes that PNNL made a thorough attempt to review the applicant's ADCIRC materials.
- The reviewer mentions that she agrees with PNNL's statements regarding realistic and accurate simulation capabilities of ADCIRC. However, the reviewer repeats her concern that ADCIRC configuration may not be sufficiently conservative because it does not consider possible changes, e.g., land cover.

- Recommendations:
  - The reviewer states the computational methods employed to quantify hurricane flood elevations were reasonable and appropriately applied.
  - The reviewer mentions that ADCIRC was appropriately configured and applied to simulate realistic hurricane flood elevations.
  - The reviewer states that selected PMH conditions do not satisfactorily consider the full range of high surge possibilities because large hurricane sizes were not considered.
  - The reviewer recommends investigation of the effects of large-radius hurricanes. The reviewer mentions that while the likelihood of a very large, very intense hurricane occurring is very small, consideration should be given to a very large, moderately intense hurricane.
  - The reviewer states that it is advisable to perform ADCIRC simulations with low friction, without wind masking, and with omission of relevant small-scale features.
- Technical adequacy of the analysis for the combined storm surge and dam breach scenario (hurricane-related hydrodynamic conditions at the MCR embankment)
  - The reviewer states that it would be valuable to validate the simplified methods used to estimate current speed from the SLOSH simulations by applying the methods to ADCIRC flood elevations and comparing with ADCIRC-simulated current speeds.
  - The reviewer recommends that ADCIRC-simulated surge time series be reviewed to confirm the applicant's finding with respect to duration of high surge and coincidence of high surge with waves directed away from the (northern) embankment.
  - The reviewer states that clarification is needed regarding how wave runup was incorporated with the ADCIRC flood elevation estimates.
- Opinion on Non-Concurrence 1
  - The reviewer does not provide her opinion because she did not review dam breach analysis methods.
- Opinion on Non-Concurrence 2
  - The reviewer states that model selection and application for quantification of hurricane flooding appears reasonable.

- The reviewer agrees with the non-concurrence on the specific issue of identification of the most severe hurricane conditions as indicated in the historical record. The reviewer states that in her opinion, the risk posed by a large storm, on the order of those seen recently in the Gulf of Mexico should be considered.
- The reviewer states that ADCIRC model and its implementation were adequately reviewed. The reviewer notes that in her opinion ADCIRC is highly suitable for use in studying extreme hurricane surge events. The reviewer also states that if the NRC intent is to have a full independent verification of all methods employed (presumably, by the applicant), then PNNL should be requested to complete some confirmatory ADCIRC simulations.

Attachment 3: Storm Surge Response to the Independent Contractor Reports
December 5, 2011

DECEMBER 5, 2011

SOUTH TEXAS NCP: SUMMARY OF FINDINGS AND CONCLUSIONS BY PANEL OF SIX INDEPENDENT SUBJECT MATTER EXPERTS

DR. HENRY JONES

Issue #2: Flood Analysis of Hurricane and MCR Breach Combination in SER Section 2.4.5

1) At the beginning of the FSAR review, the staff noted that combined events such as a combination of storm surge and MCR breach, could create a more severe flood than that of a MCR breach only. In this regard, PNNL reviewed the applicant's storm surge modeling and concluded that the applicant's probable maximum hurricane (PMH) used in the surge model is conservative. It is my opinion that the STP's PMH parameters are not conservative because the applicant's PMH is based on the NOAA's manual (NWS-23) which was developed in 1979 but has not been updated for the storms observed during the past 30 years. It is generally known that the intensity and frequency of hurricanes have been increased during the past decades. SER does not address this change adequately.

2) PNNL concluded in SER Section 2.4.5 (p. 81) that the STP's ADCIRC simulations for determining the storm surge level at the site is adequate. However, they provide no background information or any justifications on that conclusion. PNNL never review the input and output of the STP's ADCIRC model nor conduct any independent confirmatory modeling other than SLOSH simulations which were discarded by the applicant. The main reason for discarding the SLOSH model is that the STP's SLOSH model is less accurate than STP's ADCIRC model that uses finer grids with detailed topography data. Since STP adopted to use the ADCIRC model, NRC should review the STP's ADCIRC model to support the conclusion of adequacy of surge modeling and to justify the reasons why the combined event of hurricane and MCR breach is discarded for consideration.

SUMMARY OF FINDINGS

1) **NCP Issue #2, Part 1 – STP PMH parameters are not conservative due to outdated NWS-23 database (1851-1978):**

   The primary PMH parameters of concern are landfall position, central pressure, pressure differential (Δ P), forward speed, track direction and radius of maximum winds (storm radii) . *With the exception of forward speed (Luettich) and storm radii (Resio, Luettich and Irish), the SMEs found the applicant's PMH parameters conservative.*

   The subject matter experts (SMEs) are unanimous in their belief that the radius of maximum winds used by the applicant (21 nmi) wasn't the most conservative value (small hurricane). The SMEs believe that recent large storms like Katrina, Rita and Ike are not captured in the NWS-23 database. In regard to the method of addressing the issue of hurricane radii, Irish and Luettich are in agreement that the approach used by Don Resio (using ADCIRC and the JPM-OS method) already considered a storm population of large storms (greater radii of maximum winds).

Luettich disagreed with PNNL's conclusion that a "fast moving" hurricane maximum storm surge at the site. Both PNNL SLOSH and Resio ADCIRC results confirm that Luettich was correct in that an intense, "slow moving", large radii hurricane will produce the maximum storm surge at the STP site.

Resio, after reviewing the applicant's PMH parameters, looking at the tradeoffs in their use of a more intense storm (with no decay before landfall) and smaller radii hurricane, concluded that the "applicant's values obtained via their ADCIRC simulations will likely be consistent with values obtained via a more modern approach

Irish's main concerns were the omission of large hurricane radii, non-address of climate change and non-conservative (applicant used realistic bottom friction) setup of ADCIRC with regards to bottom characteristics. However, Irish also stated that the applicant's central pressure deficit is "conservative, or extreme, in the sense that it is significantly higher, about 30%, than the largest central pressure deficit appearing in the historical record for U.S. landfalls along the Gulf of Mexico since the 1940s... Conservatism is also added by the assumption that the applicant limits hurricane decay as the hurricane approaches the coast."

2) **NCP Issue #1, Part 2 – Review of the applicant's ADCIRC model and the plausibility of a storm surge and MCR Breach combined event**

Review of Applicant's ADCIRC Model: In regard to the validity of the ADCIRC model, Irish concludes that *"it appears that the model was applied in a reasonable manner."* In the framework for PMH/PMSS estimation, Resio *"concurs with the staff's decision on the licensing at this site."*

Combined Event - Storm Surge causing a MCR Breach: Irish, Patev and Baecher don't address this issue.

BUREC concurred that a MCR embankment breach due to PMSS loading is not a credible failure because the presumed loading primarily attacks the toe berm and not the main body of the MCR. In addition, the duration of the loading is short and the vegetal cover and embankment soils are expected to prevent erosion into the embankment.

Luettich concludes that the MCR (northern embankment) should not be vulnerable to significant wave damage associated with the PMSS because the surge inducing winds would not come from a northerly direction.

Resio concurs with the staff's conclusion that PMSS wave effects would not lead to a MCR embankment breach.

DISCUSSION

In response to Dr. Ahn's insistence on a scenario where storm surge causes a MCR embankment failure, the applicant applied the ADCIRC model to find a conservative (not most conservative) and realistic PMSS using current NRC guidance. The desired outcome, which was accomplished, was to show that the PMSS would not exceed 28-29 ft MSL (MCR northern embankment), thus excluding the PMSS/MCR breach scenario.

Resio and Irish were in agreement in that the applicant's ADCIRC model was applied in a reasonable manner. Although the applicant uses a non state-of-the-art storm parameter methodology (PMH/NWS-23), the values obtained via their ADCIRC simulations are consistent with values obtained via a more modern approach.

PMSS/MCR Embankment Failure

Based on the above conclusions, the applicant's ADCIRC PMSS of 29.3 ft is reasonable and conservative. In agreement with Luettich, the PMSS/MCR breach issue is moot because the storm surge barely reaches the northern embankment. In addition, basic meteorological and oceanographic physics apply:

1) Regardless of the PMSS height, the meteorological conditions that would cause erosive wind waves and currents are counter to the conditions for significant PMSS. Waves travel in the direction of the wind (Luettich, Resio and BUREC). The wind conditions for a conservative PMSS would be directed towards the northwest (away from the northern MCR embankment).

2) Current velocity is a function of wind speed not storm surge height. In additiom, maximum current velocities will be in a direction 15-45% to the right of the wind direction (Northern Hemisphere). Thus, regardless of the sustained wind velocity/maximum current velocities, the currents will be directed away from the MCR northern embankment.

3) The winds will not be continuous due to land interaction/storm movement (decay) and blockage by the MCR.

4) The MCR embankment vegetation and clay content of the underlying zone B materials would mitigate erosion.

Climate Change and increasing storm intensity

Recent research on Atlantic hurricanes and climate change has focused on whether the increase in hurricane activity in the basin since the 1970s portends future large increases in a warming climate. In an analysis of projected climate changes over the tropical Atlantic region during the 21st century derived from 18 different climate models developed for the IPCC Fourth Assessment Report, a notable finding was that the vertical wind shear (the difference in wind direction and speed between the lower and upper atmosphere) is projected to increase across much of the Caribbean in the warmer climate, <u>a factor that tends to suppress tropical storm and hurricane development and intensification.</u>

Validity of NWS-23 (1851-1978)

The NRC and other agencies such as NOAA, FEMA and USACE are focused on hurricanes that <u>make landfall</u>. The frequency of hurricanes in along any fifty mile segment of the Texas coast is one about every six years. Annual probabilities of a strike along a fifty mile segment range from 31% at Sabine Pass to 41% around Matagorda Bay. The annual average occurrence of a tropical storm or hurricane per year is 0.8, or 3 per every 4 years

There were fifty-four hurricanes that impacted Texas between 1851 and 2008 with18.5% occurring outside the NWS 23 reporting period. No hurricane greater than category 4 has ever made landfall in Texas. All category 4 hurricanes occurred within the NWS 23 reporting period. The applicants PMH is a category 5 (184 mph). At 887 mb, the applicant's PMH central pressure is only 2 mb above the lowest central pressure for NWS 23 (885 mb) and 7 mb above the projected theoretical lowest central pressure for the Atlantic (880 mb). In addition, the applicant's wind speed of 184 mph is 4 mph greater than the highest recorded hurricane speed in Texas (180 mph in 1970 at Port Aransas). The highest wind speed at landfall in U.S./world history is 190 mph (Hurricane Camille of 1969). Note that the applicant didn't use any decay, thus bringing a storm ashore that is below the world record by 6 mph.

The applicant's 29.3 ft (29.95 ft with additional 0.65 ft runup) is approximately equal to the highest storm surge in U.S. history (30 ft, Katrina, 2005). Resio's simulations uses large storm s(large hurricane radii) with recurrence probabilities of 10E-7 to 10E-12 based on central pressure difference and radius of maximum winds. Resio also used the most conservative parameters to produce a "screening" storm for the site (50+ ft).

As Resio predicted in his research, increasing storm size reaches its limit around 30-42 nmi. Following current NRC guidance and replacing his screening parameters (with the exception of wave runup and setup) with site specific values, the 10E-7 to 10E-12 storms will produce storm surges no greater than 45 ft. As RG 1.59 allows for more realism (e.g., bathymetry), the introduction of realistic friction, site specific runup, and a higher resolution grid (e.g., applicant's ADCIRC bathymetry) would most likely reduce the PMSS to 40 ft or less (below the MCR breach DBF of 40 ft msl).

As previously mentioned in the SER, it is the high resolution grid (resolves the Matagorda levee) that shows the blockage of the PMSS by the levee, thus preventing the storm surge from reaching the site (34 ft msl).

CONCLUSION AND RECOMMENDATION

I recommend including the overall findings of the NCP reports into the STP SER by reference and providing additional comments in Sections 2.4.4 (Dam Failure) and 2.4.5 (Surge and Seiche): ADCIRC model verified by three technical experts; PMSS of 29.93 ft is conservative/reasonable; climate change findings by IPCC regarding tropical cyclone intensity, and; MCR breach caused by PMSS not probable.

Attachment 4: STP NC Review Storm Surge – Irish
November 29, 2011

Independent Technical Review:
South Texas Plant Units 3 and 4 Storm Surge Analysis

Jennifer L. Irish, Ph.D., P.E., D.CE

Final Report
October 27, 2011

Introduction

The purpose of this report is to review documentation and conclusions leading to non-concurrences arising during the Nuclear Regulatory Commission's (NRC) independent peer review of documentation provided with a license application for the South Texas Plant. The primary outcome of this review is to give an opinion regarding the validity of the non-concurrences, specifically with respect to standard practices for assessing extreme hurricane surge, hurricane-induced hydrodynamic forcing at the Main Cooling Reservoir (MCR) Embankment, and final conclusions. The two relevant non-concurrences are: (1) breaching analysis for the MCR and (2) combined flood analysis for hurricane with a MCR breach. Specifically, these non-concurrences postulate that the design basis flood level is not sufficiently conservative for two main reasons: (1) the breaching analysis is flawed, (2) the probable maximum hurricane parameters were selected based on dated information, and (3) the hurricane surge analysis made with ADCIRC was not sufficiently vetted.

As part of this report, both the applicant's storm surge and related breaching analysis, hereafter Safety Analysis Report (SAR), and the subsequent independent peer review, hereafter Safety Evaluation Report (SER), will be generally reviewed for technical merit, specifically with respect to the evaluation of design flood levels, for storm surge. A more detailed review will be presented for items specifically related to the non-concurrence items. In review of the non-concurrence items, the formal Response made by the Pacific Northwest National Laboratory (PNNL) will also be reviewed. In reviewing all documents, emphasis will be placed on assessing the technical soundness of (1) selection of maximum probable hurricane parameters, (2) simulation of storm surge, e.g., with ADCIRC, and (3) assessment of hurricane surge impact conditions at the MCR embankment. The aim is to give an opinion about the technical adequacy of (1) the storm surge analysis and (2) the analysis for the combined storm surge and dam breach scenario, specifically in regard to quantification of hurricane-related hydrodynamics at the MCR Embankment.

This report is organized into six sections as follows. The introduction provides the framework and context for the review. This section is followed by a summary of the non-concurrence items to be assessed. The following two sections provide a general review the technical merit of the storm surge and related hydrodynamic conditions at the MCR Embankment made in the SAR and SER, respectively. Reviews of the non-concurrences and of the PNNL Response are then presented. In the last section, all key findings during this review are summarized, and opinion statements are made with regard to the validity of hurricane hydrodynamics related non-concurrences.

Summary of Safety Evaluation report non-concurrences 1 and 2

During the Nuclear Regulatory Commission's independent peer review of documentation provided with a license application for the South Texas Plant, two non-concurrences arose in regard to surge hazard, or design basis flood level. Specifically, the non-concurrences are motivated by the opinion that the design basis flood level is not sufficiently conservative, or severe.

The items raised in Non-concurrence 1 are primarily related to the choice and application of the breaching methods. The specific criticisms raised in this non-concurrence will not be addressed here. The items raised in Non-concurrence 2 are specifically related to the suitability of the design basis flood level. In the non-concurrence, it is postulated that the probable maximum hurricane conditions selected by the applicant are not sufficiently conservative because recent storms are not considered. The second criticism raised in Non-concurrence 2 relates to the conduct of the independent review; in the non-concurrence it is claimed that a rigorous review of the ADCIRC simulations was not carried out. In the review below, focus is given to the merit of the selection of probable maximum hurricane conditions and of the application of ADCIRC.

Review of Safety Analysis Report

In this section all information provided by the applicant related to assessment of hurricane surge hazard and the risk of MCR breach due to hurricane forcing is reviewed. Specifically, the following documents were considered:

- SAR2.4S.4 (as it relates to the MRC breach due to hurricane forcing)
- SAR2.4S.5
- ML081970231
- ML082560248
- ML102100047
- ML103330369
- ML082490086
- ML082530449
- ML092610376

The sections below provide specific review of condition selection for probable maximum hurricane surge, simulation of hurricane surge, consideration of other coastal water level contributions, and an assessment of the validity of the results for of the overall suitability, as well as the specific suitability for MRC breach analysis.

Probable Maximum Hurricane

The storm surge analysis is based on probable maximum hurricane (PMH) conditions. The applicant considered the relevant meteorological parameters of landfall position, central pressure deficit, storm radius, forward speed, and track angle. Probable storm radii and forward speeds were determined by considering the historical record; this is a technically sound approach. The forward speed range selected by the applicant appears suitable. However, the applicant's upper radius to maximum winds of 21 nmi (39 km) is not representative of the upper limit for radius at landfall for landfalls along the U.S. Gulf of Mexico coastline. Since 1940, there have been 10 hurricanes with radius to maximum wind larger than this upper limit (U.S. Army Corps of Engineers 2006); of note are Hurricane Carla in 1961 (56 km), Hurricane Betsy in 1965 (74 km), and Hurricane Ike in 2008 (estimated as 50 km based on curve fit to data in Powell and Reinhold(2007)). While some of these storms are within the 30-year record considered by the applicant, recent large storms like Hurricanes Katrina, Rita, and Ike are not. In fact it was these recent storms that revealed the important role of storm size in surge generation over shallow shelves (Irish et al. 2008). With all else being equal, a large radii storm will lead to higher surges than a small radii storm. It is the combination of high winds and size that govern surge generation, and it does not necessarily follow that the storm with the strongest maximum winds will generate the largest surge (Irish and Resio 2010). The South Texas Plant's location, in the vicinity of Matagorda Bay, is fronted by a moderately wide continental shelf. Additionally, recent studies have shown that this area is somewhat more vulnerable to high surge generation, with respect to surrounding areas, due to the change in orientation and significant widening of the continental shelf to the northeast (Irish et al. 2009; Irish et al. 2011). However, there is a correlation between storm size and storm central pressure, where more intense storms often are of smaller size.

The extreme-value of central pressure deficit was assumed constant at 133 mb. This central pressure deficit is conservative, or extreme, in the sense that it is significantly higher, about 30%, than the largest central pressure deficit appearing in the historical record for US landfalls along the Gulf of Mexico since the 1940s (103 mb [1013-910mb] for Hurricane Camille). The applicant's selected value for central pressure deficit is close to the current-day minimum probable intensity for the Gulf of Mexico, on the order of 133 mb (Irish et al. 2008; Tonkin et al. 2000). Conservatism is also added by the assumption that the applicant limits hurricane decay as the hurricane approaches the coast. The applicant does not address potential future hurricane intensification and its implications.

There is only a limited discussion of storm track selection, namely landfall location, within the SAR itself. In the application of SLOSH, it appears that a series of approximately head-on tracks were considered (see Fig. 2.4S.5-7) while for the application of ADCIRC, in one of the presentations provided, it appears that one approximately head-on track was considered. In the applicant's response to RAI.02.04.05-10, the applicant states that "numerous combinations of hurricane track directions and landfall locations" were considered while in the applicant's response to RAI.02.04.05-11, the applicant states that three landfall positions were considered on three separate track orientations. Here, the applicant also indicates that the landfall positions were spaced one radius to maximum winds apart, but it is not clear which of the three radii was used. In their discussion of surge results, the applicant indicates a nonlinear trend in surge with respect to track; this is not surprising for storms making landfall close to the point of interest. In the vicinity of Matagorda Bay, we have found that maximum surge occurs when the storm is located on average 0.87 times the pressure radius to the left of the location of interest (Irish et al. 2009); this is close to the 1 radius to maximum winds assumed by the applicant.

The hurricane frequency reported by the applicant of about 33% chance of landfall in any given year is consistent with the historical record. No consideration is given to possible future changes in hurricane frequency.

Storm Surge Analysis Approach

The applicant employed three different models for hurricane surge simulation: SURGE and SLOSH in the main SAR and ADCIRC in their response to NRC review. All three models solve for momentum conservation of long waves. A main limitation of SURGE is that it only considers surge generation along a single transect, thus ignores two-dimensional spatial variations in surge. As such, this is an over simplified model for hurricane surge. Because of the offshore bathymetry in the vicinity of Matagorda Bay is not uniform alongshore, it is possible that this model will miss potential surge amplification due to changes in continental shelf width to the north. In application of this model, the applicant compensates for possible errors introduced by the one-dimensional assumption via calibration of the friction factor. The applicant applied SURGE using their developed PMH conditions and concluded that highest surges occur for large, fast-moving storms.

The SLOSH model on the other hand, currently in regular use by NOAA for hurricane surge forecasting, does consider the two-dimensional spatial variability in surge response. This is an improvement over the SURGE model. However, a main drawback of SLOSH is that spatial resolution is typically sacrificed in the interest of computational speed; the result of coarse resolution is loss in accuracy in the flood elevation predictions at a specific location. The applicant applies the SLOSH database for maximum of the maximum envelop of water (MOM), and specifically considers the largest intensity storm set, Category 5. The applicant notes that the MOM for the Category-5 class of storm yields lower flood elevations than the SURGE, thus an adjustment to the SLOSH results was applied. This adjustment is made empirically via a polynomial fit to the SLOSH data based on central pressure deficit. Since wind speed is related to the square root of the central pressure deficit, this approach appears to be reasonable. However, it appears that a very limited number of data points were used, thus a third-degree polynomial may not be justified. The applicant confirmed the conservativeness of this approach by comparing to ADCIRC. Based on this analysis, the applicant concludes that the PMH flood elevation is not the critical high flood elevation for design. In this analysis, the applicant again concluded that large, fast-moving storms generated highest surges.

The ADCIRC model is widely accepted as an accurate model for hurricane surge. Throughout the last two decades, this model has continually undergone improvements and advancements to incorporate relevant physical processes for surge generation. Applied at high resolution, ADCIRC represents the state-of-the art for hurricane surge simulation. In their responses to RAI.02.04.05-10 and RAI.02.04.05-11, the applicant applied ADCIRC along with new modeling with SLOSH to confirm earlier conclusions with SURGE and SLOSH. As explained in RAI.0-2.04.05-11, the application of both ADCIRC and SWAN, inclusive of the Holland model for meteorological forcing, appear reasonable and follow current standards for these models. However, some of the conservatism considered in applying both SURGE and SLOSH is not carried forward with this modeling. Rather, a more realistic description of small-scale land characteristics is applied. Specifically, variable bottom friction and vegetation wind masking are employed, and small-scale topographic barriers are included. These three processes, while leading to a more realistic representation of current-day conditions, will most likely lead to lower flood elevation estimates at many locations, with respect to the lower friction analyses included in SLOSH and SURGE. The combination of these processes, along with significant differences in spatial resolution, is likely what leads to the significantly lower flood estimate provided by ADCIRC.

Analysis of Other Coastal Flood Level Contributions

The applicant, in their response to RAI.02.04.08-1, gives a description of the methods used to quantify wave height and runup at the MRC embankment based on the U.S. Army Corps of Engineers methods. These methods are considered standard for simple wind-wave height and runup estimation. The runup, calculated to be the vertical excursion above the mean long wave elevation (surge, tides, etc.), is added to the surge estimate from SURGE and SLOSH. This is a conservative approach for SLOSH, since this model includes an empirical correction for wave setup; the addition of runup in this way effectively double-counts wave setup. While the applicant provides a discussion of how SWAN was applied to assess wave setup, it is unclear how wave runup is calculated and included in the ADCIRC simulations. The applicant implies this is directly computed; however, ADCIRC cannot be used to compute wave runup, a short wave process.

The applicant argues the validity of the surge analysis work in their comparison of the 6 modeling approaches employed (various combinations of the three models and assumptions) as presented in the applicant's response to RAI.02.04.05-10. All but one method, ADCIRC, give comparable magnitudes. The ADCIRC result is considerably lower, likely due to the more realistic assumptions made in implementing this model.

The applicant considers both additional elevation due to astronomical tides and wave setup. Astronomical tides are considered to be additive, and this is a reasonable approach. The applicant determined wave setup to be small and within the "conservatism of the approach", but the applicant does not specify the computed wave setup amount for SLOSH applications. On the other hand, a thorough explanation is provided on how SWAN was integrated with ADCIRC to directly compute wave setup during execution of ADCIRC.

The applicant considers possible future changes in flood conditions due to sea-level rise only. This is accomplished by assuming the historical sea-level rise rate continues into the future. No consideration is given to potential future sea-level rise acceleration.

Analysis Specific to a Surge-Induced MCR Breach
The applicant specifically addresses the scenario of a hurricane surge induced MCR breach in their response to RAI.02.04.05-10 and RAI.02.04.05-11. Here, they argue that the timing of highest surge is relatively small, such that the risk of breaching is very low; this is a reasonable assumption, but it would be useful to see simulated flood time series at the site. The applicant also argues that dominant wind direction at the time of highest surge would be pointing away from the site. This is a reasonable assumption if high surge coincides with landfall. It is not clear that the applicant has reviewed flood elevation time series to confirm that this is indeed the case. The applicant acknowledges that a possible risk factor is strong currents. Current magnitude was estimated from SLOSH flood elevations using three methods. These three methods appear to be reasonable engineering approximations of current speed. However, no direct comparison between the methods and simulated currents is provided; this could have been accomplished via the use of ADCIRC and would lend credibility to the simplified methods applied to the SLOSH water levels.

The applicant specifically addresses wind-wave setup and runup at the MCR embankment in their response to RAI.02.04.08-1. The methods employed follow standard practice for simple engineering calculation, and results appear reasonable. It is recommended, however, that during the design phase, a more rigorous consideration of wave conditions and runup be executed. This could be accomplished through full-plane spectral wave modeling coupled with a local Boussinesq or Navier Stokes model.

Validity of Surge Analysis Results
The applicant has considered three progressively more complex models for assessing hurricane flood elevation. In general the surge modeling approach appears to be thorough, and reasonably conservative in most aspects.

My primary concern regarding the validity of the overall surge analysis results for assessing the worse case surge event is in the omission of large hurricane radii. While very large hurricanes typically do not support very low central pressures, it would be prudent to additionally consider the impact of aN intense large hurricane.

A secondary concern is in regard to the non-conservative setup for ADCIRC, with regards to bottom characteristics. It would be useful to demonstrate that ADCIRC yields flood levels at or below those estimated with SURGE and SLOSH when non-conservative bottom friction, wind masking, and small-scale feature assumptions are applied.

Other concerns that warrant some clarification and discussion are:

- Details regarding the wave runup calculation and implementation with ADCIRC flood estimates.
- Some discussion of the potential implications of possible future climate change impacts and possible acceleration in sea-level rise would be beneficial for understanding overall future flood risk.

Suitability of Surge Results for MCR Breach Analysis

With regards to the hydrodynamic loading at the MRC breach, namely flood elevation, wave height, wave runup, and currents, I have two additional concerns beyond those raised in the previous section. First, in regard to currents, it would be valuable to validate the simplified methods used to estimate current speed from the SLOSH simulations by applying the methods to ADCIRC flood elevations and comparing with ADCIRC-simulated current speeds.

Second, in regard to high surge and wave timing, it would be prudent to review ADCIRC-simulated surge time series at the MCR Embankment to validate the applicant's conclusions regarding short high surge duration and coincident high surge and wave direction.

Review of Safety Evaluation Report

In this section all information provided by the independent review team, the Pacific Northwest National Laboratory, related to assessment of hurricane surge hazard and the risk of MCR breach due to hurricane forcing is reviewed. Specifically, the following documents were considered:

- SER2.4S.4.4 (as it relates to the MRC breach due to hurricane forcing)
- SER2.4S.5.4

The sections below provide specific review of PNNL's conclusions regarding the condition selection for probable maximum hurricane surge, simulation of hurricane surge, consideration of other coastal water level contributions, and an assessment of the validity of the results for of the overall suitability, as well as the specific suitability for MRC breach analysis. Critique of new analyses is also included below. It should be noted that all of the items raised as points of concern by PNNL was subsequently addressed by the applicant; the applicant's responses were reviewed in the above section Review of Safety Analysis Report.

Probable Maximum Hurricane

PNNL reaffirmed the applicant's selection of the probable maximum hurricane conditions by repeating their analysis. However, they did request justification from the applicant regarding the omission of recent historical hurricanes from the analysis to determine these extreme conditions. PNNL indicates in the SER that "currently available information" was reviewed to select conditions for testing. They arrived at values similar to those employed by the applicant. The upper value of hurricane size selected by PNNL coincides with that selected by the applicant. PNNL, however, does note that other contemporary work suggests use of larger hurricane sizes.

Storm Surge Analysis Approach

PNNL repeated the applicant's analysis with SURGE and arrived at similar results. However, PNNL criticized the applicant's use of an extrapolation approach for comparison with SLOSH database results as not being conservative enough. Independent assessment by PNNL with SLOSH confirmed this hypothesis. In the independent review, the SLOSH model was applied with hurricane parameters consistent with those employed by the applicant and along seven tracks bounding the study site; three approach angles were considered. In response, the applicant employed a more rigorous surge modeling approach with ADCIRC, as reviewed above.

With regard to the impact of hurricane size on surge generation, PNNL compared surge estimates from Resio (2009), which consider larger storm sizes up to pressure radii of 117 km (63 nmi). They state that based on this analysis, surge inundates the site. While from the Resio (2009) presentation provided it is evident that these very large storms result in high surges, it is not clear which simulation corresponds to which track; the highest surge value for a large storm at the site appear to be due to storm 40 and be between 9 and 10 m above the model datum.

PNNL concurs with the implementation of ADCIRC, coupled with SWAN, by the applicant. However, PNNL did not confirm the application of ADCIRC by independent simulation, as was carried out for SLOSH. PNNL noted the added benefit of spatial detail for accurate surge simulation. No mention was made regarding the non-conservative assumption of current-day friction, wind masking, and small-scale features. Thus, the concern still remains that this scenario may not be maintained in the future due to future land development or damages incurred during storms.

Analysis of Other Coastal Flood Level Contributions

Regarding wave height and wave runup calculation, PNNL repeated the applicant's analysis following standard U.S. Army Corps of Engineers methods for simple calculation. Their analysis appears to yield smaller values, thus the applicant's values are more conservative. Also in regard to wave runup, PNNL also noted that wave runup is not included in the ADCIRC output, then go on to say that the applicant's estimate of runup could be added to the ADCIRC water level. This would be an appropriate conservative approach, where wave setup is double-counted.

Validity of Surge Analysis Results

PNNL followed procedures similar to those employed by the applicant; however, they did not execute ADCIRC to complete an independent verification. They further explored the scenario where a larger hurricane occurs; however, they do no more than state that some evidence supports "inundation." A failing of this review by PNNL appears to be in not exploring further the hazard posed by surge generated by large hurricanes.

Suitability of Surge Results for MCR Breach Analysis
PNNL identified the possibility of extreme hurricane surge being a trigger for and MCR breach; the additional analyses provided by the applicant were viewed by PNNL to satisfactorily address conditions associated with this potential scenario. There are no additional concerns regarding this analysis, beyond those mentioned previously.

Review of Safety Evaluation Report non-concurrences 1 and 2

The document titled "Enclosure 1: Re-Analysis of MCR Breach Flood" does not specifically address criticisms raised in regard to hurricane forcing at the MCR embankment with the non-concurrence comments in the document titled "Enclosure 2: PNNL's Calculation Package – Commented and corrected by Dr. Hosung Ahn" only addresses breach calculation procedures. Thus no review comments related specifically to the selection of probable maximum hurricane conditions and development of hurricane-related forcing at the MCR embankment can be made.

PNNL provided a response to the non-concurrences. First, in regard to selection of a worse case probable maximum hurricane event, PNNL makes two points. First, they reiterate the findings from their independent assessment of the 30-year historical record to quantify hurricane statistics. In this response, there is no specific mention on their findings regarding hurricane size.

Second, PNNL discussed their interpretation of "the most severe of the natural phenomena" as it pertains to the MCR breach analysis. In the quoted text, it is noted that consideration should be given for the most severe phenomena historically reported. With this statement, it can be concluded that the NRC does not require consideration of future climate conditions. PNNL argues that the applicant has sufficiently considered maximum floodwater elevation by adding a 0.1 m margin to the estimated flood elevation of 39.9 ft; PNNL deems this flood elevation estimate conservative. The computational methods employed in arriving at the 39.9 ft flood elevation appear sound, and given the conservatism of this analysis, a 0.1 ft margin appears suitable for initial planning purposed. However, my concern still remains that large hurricanes, which appear in the historical record and have occurred recently (e.g., Katrina [2005], Rita [2005], and Ike [2008]), have not been considered in this analysis. No new concerns arose in my review of this response.

Third, PNNL also provided a response to the non-concurrence concern that inadequate review of ADCIRC was made. It appears that, while simulations were not repeated, PNNL made a thorough study of the information provided by the applicant and supplemented this information with additional literature and information available on the use of ADCIRC. ADCIRC is a computationally intensive model requiring specific expertise and can thus very expensive to execute. It is my opinion that PNNL made a thorough attempt to review the applicant's ADCIRC materials. While I agree with PNNL's statements regarding the realistic and accurate simulation capabilities of ADCIRC as applied in this study, I am still concerned that this configuration may not be sufficiently conservative since it does not consider possible changes, for example in land cover.

Summary and Recommendations

Technical adequacy of the storm surge analysis

Overall, the computational methods employed to quantify hurricane flood elevation were reasonable and appropriately applied. Specific review of the ADCIRC analysis indicated that this model was appropriately configured and applied to simulate realistic hurricane flood elevations. However, it is my opinion that the selected probable maximum hurricane conditions do not satisfactorily consider the full range of high surge possibilities because large hurricane sizes are not considered. Below is a summary of my key concerns about the storm surge analysis.

One primary concern arose during review of the storm surge analysis, and is related to the selected hurricane size range for the probable maximum hurricane. Since surge generation in the Matagorda Bay region is sensitive to hurricane size, it is recommended that the potential impact of larger radii hurricanes on flood conditions be assessed. While the likelihood of a very large, very intense hurricane occurring is very small, consideration should at least be given to a very large, moderately intense hurricane, if not a very intense hurricane, with respect to the assumed worse-case moderately large, very intense hurricanes already considered in this study. A more conservative approach would be to consider a very large, very intense hurricane. Omission of realistic large hurricane sizes, like those appearing in the historical record, could lead to an underestimation of the surge hazard. It is worth noting that both Hurricanes Carla and Betsy are within the dataset selected for analysis by the applicant; this radii dataset should be reviewed to assess potential inconsistencies with current reported radii values and/or implementation in the various wind models.

One secondary concern arose during review of the storm surge analysis. This secondary concern is in regard to the non-conservative setup used in the ADCIRC simulations. It is advisable that a subset of simulations be carried out with low friction, without wind masking, and with the omission of relevant small-scale features if the intent of the ADCIRC simulations is (1) to ascertain the worse-case future flooding and (2) to directly compare to SLOSH and SURGE simulations, which make these conservative assumptions. Such an analysis would take into consideration, in a conservative manner, of future land use changes (e.g., wetlands or forest converted to residential) and failure of small-scale topographic barriers such as levees and roadways, thus leading to a worse case upper flood level for comparison with SURGE and SLOSH.

Finally, I have one minor concern regarding the storm surge analysis, related to consideration of implications from possible future climate change. While sea-level rise is considered by extrapolating historical rates, no consideration is given to possible acceleration in sea-level rise. Similarly, no consideration is given for possible increases in hurricane intensification and for possible changes in hurricane frequency. It would be worth adding at least a short, qualitative discussion regarding these future possibilities.

Technical adequacy of the analysis for the combined storm surge and dam breach scenario (hurricane-related hydrodynamic conditions at the MCR Embankment)

Two secondary concerns arose during review of hydrodynamic conditions at the MCR embankment. One is in regard to calculation of the current speed at the MCR embankment. It would be valuable to validate the simplified methods used to estimate current speed from the SLOSH simulations by applying the methods to ADCIRC flood elevations and comparing with ADCIRC-simulated current speeds. This would lend confidence to the method as well as help to quantify possible uncertainty in the adopted approach.

The second secondary concern is in regard to the assessment of the timing of high surge and waves at the MCR Embankment. It is recommended that ADCIRC-simulated surge time series be reviewed to confirm the applicant's findings with respect to duration of high surge and coincidence of high surge with waves directed away from the embankment.

I have one minor concern regarding the hydrodynamic conditions at the MCR embankment, and it relates to the calculation of wave runup in the application of ADCIRC. Clarification is needed regarding how wave runup was incorporated with the ADCIRC flood elevation estimates. It is not clear that any additional calculation beyond model output was made; this is incorrect since ADCIRC cannot be used to simulate short-wave processes like runup.

Opinion on Non-Concurrence 1
As discussed earlier, the specific criticisms raised in Non-concurrence 1 are related to the breaching analysis methods and their application. These methods and their application were not reviewer here, thus no opinion can be given.

Opinion on Non-Concurrence 2
As stated above, the model selection and application used to quantify hurricane flooding appear reasonable. However, I am in agreement with the non-concurrence on the specific issue of identification of the most severe hurricane conditions, as indicated in the historical record. It is my opinion that the risk posed by a large storm, on the order of those seen recently in the Gulf of Mexico, should be considered in this analysis. Please see the discussion in "*Technical adequacy of the storm surge analysis*" above for specific recommendations regarding hurricane size.

The second issue raised in Non-concurrence 2, related to hurricane forcing at the MCR Embankment, is adequate review of the ADCIRC model and its implementation. It is my opinion that through this third review by the other two surge experts and myself that this non-concurrence item will likely be resolved. However, if it was the NRC's intent to have a full independent verification of all methods employed, PNNL should be requested to complete some confirmatory simulations with ADCIRC. It is my opinion that ADCIRC is highly suitable for use in studying extreme hurricane surge events, and it appears that the model was applied in a reasonable manner. I did raise a secondary concern above, with regard to the conservativeness of the model setup, and it may be worth conducting a sensitivity to these parameters; see *Technical adequacy of the storm surge analysis* above for specific recommendations.

References

Irish, J. L., and Resio, D. T. (2010). "A hydrodynamics-based surge scale for hurricanes." *Ocean Eng.*, 37(11-12), 1085-1088.

Irish, J. L., Resio, D. T., and Cialone, M. A. (2009). "A surge response function approach to coastal hazard assessment. Part 2: Quantification of spatial attributes of response functions." *Natural Hazards*, 51(1), 183-205.

Irish, J. L., Resio, D. T., and Ratcliff, J. J. (2008). "The influence of storm size on hurricane surge." *Journal of Physical Oceanography*, 38(9), 2003-2013.

Irish, J. L., Song, Y. K., and Chang, K.-A. (2011). "Probabilistic hurricane surge forecasting using parameterized surge response functions." *Geophysical Research Letters*, 38, L03606.

Powell, M. D., and Reinhold, T. A. (2007). "Tropical cyclone destructive potential by integrated kinetic energy." *Bulletin of the American Meteorological Society*, 88(4), 513-526.

Tonkin, H., Holland, G. J., Holbrook, N., and Henderson-Sellers, A. (2000). "An evaluation of thermodynamic estimates of climatological maximum potential tropical cyclone intensity." *Mon. Wea. Rev.*, 135, 746-764.

U.S. Army Corps of Engineers. (2006). "Performance evaluation of the New Orleans and southeast Louisiana hurricane protection system draft final report of the Interagency Performance Evaluation Task Force." U.S. Army Corps of Engineers.

Attachment 5: STP NC Review Storm Surge – Leuttich
November 22, 2011

External Review of non-concurring issues on the Safety Evaluation Report for the South Texas Project Combined License Application.

Prepared by: Rick Luettich

revised: November 21, 2011

I. Review Coverage and Organization

This review report addresses issue #2: *Flood Analysis of Hurricane and MCR Breach Combination in Safety Evaluation Report (SER) Section 2.4.5* identified in the non-concurrence summary report by Dr. H. Ahn, dated June 8, 2011. The text of Issue #2, as presented in the non-concurrence summary report, is:

1) At the beginning of the FSAR review, the staff noted that combined events such as a combination of storm surge and MCR breach, could create a more severe flood than that of a MCR breach only. In this regard, PNNL reviewed the applicant's storm surge modeling and concluded that the applicant's probable maximum hurricane (PMH) used in the surge model is conservative. It is my opinion that the STP's PMH parameters are not conservative because the applicant's PMH is based on the NOAA's manual (NWS-23) which was developed in 1979 but has not been updated for the storms observed during the past 30 years. It is generally known that the intensity and frequency of hurricanes have been increased during the past decades. SER does not address this change adequately.

2) PNNL concluded in SER Section 2.4.5 (p. 81) that the STP's ADCIRC simulations for determining the storm surge level at the site is adequate. However, they provide no background information or any justifications on that conclusion. PNNL never review the input and output of the STP's ADCIRC model nor conduct any independent confirmatory modeling other than SLOSH simulations which were discarded by the applicant. The main reason for discarding the SLOSH model is that the STP's SLOSH model is less accurate than STP's ADCIRC model that uses finer grids with detailed topography data. Since STP adopted to use the ADCIRC model, NRC should review the STP's ADCIRC model to support the conclusion of adequacy of surge modeling and to justify the reasons why the combined event of hurricane and MCR breach is discarded for consideration.

The primary documents evaluated for this review are:

SER Section 2.4S.5 – Probable Maximum Surge and Seiche Flooding
Response to RAI 02.04.05-10
Response to RAI 02.04.05-11

Other material provided by the NRC, including presentations (Bailey, 8/31/2010, Resio, 11/18/2009; Vickery, 2009), ADCIRC input files, the code for ADCIRC's wind model as modified to use the SLOSH radial wind profile and other documents were also reviewed to provide additional context and information in support of the evaluation of the primary documents.

This document presents Summary Findings in Section **II** followed by a detailed evaluation of the primary documents in Sections **III-V**. Each relevant section of the primary documents is followed by this reviewer's comments and recommendations regarding the document content and any additional analysis that should be conducted for the STP site.

II. Summary Findings

It is my overall opinion that based on the material provided by applicant in the SER and the associated responses to RAIs, the non-concurrence conclusion reached by Dr. Ahn is justified. While I believe that ADCIRC is the appropriate model to use to evaluate the PMSS, I do not believe that the applicant has applied it in a conservative manner. Sections **III-V** provide a detailed set of comments that support this opinion, however, the primary reasons are summarized below.

1.) I believe that the PMH radius to maximum winds is underestimated in NWS 23 since the calculated values are less than several recent storms that have made landfall in the Gulf of Mexico. Storm surge is very sensitive to storm size and therefore, it is critical that larger RMW values are considered when determining the PMSS.

2.) While the applicant cites historical storms that have been used to validate ADCIRC along the Texas coast, none of this material has been published or was provided to the reviewers. Thus, it is not possible to assess ADCIRC skill (or overall uncertainty) in the vicinity of the STP site as a part of this review.

3.) I do not believe that ADCIRC runs using a moderately sized, fast moving storm that ramps up from category 1 to category 5 over the 12 hrs prior to landfall will lead to an accurate representation of the PMSS at the STP site. Rather, I believe this conclusion is based on flawed SLOSH runs and that the justifications for this conclusion provided in Response to RAI 02.04.05-11 are incorrect.

4.) Several evaluations of the PMSS were performed using SLOSH (SLOSH MOMs analysis, NRC staff runs, applicant runs, Vickery 2009). Due to a number of serious issues with the implementation and use of SLOSH at this location, I believe that these results are unreliable and should not be used.

5.) The Resio (2009) study of storm surge in the vicinity of the STP appears to be more thorough than those conducted by the applicant and the NRC staff. While the Resio (2009) powerpoint presentation of this work was short of details and context, it indicates a maximum ADCIRC surge value of approximately 30 ft that presumably represent the PMSS. However, after the Resio study was completed, ADCIRC was substantially re-calibrated in the Gulf of Mexico based on studies of hurricane Ike. It is not clear how this re-calibration would affect the Resio results. I recommend repeating one or two of the runs that resulted in the greatest surge levels near the STP site using the re-calibrated ADCIRC. If the results are comparable to Resio's original results, it is likely that these represent the PMSS at the site. If the re-calibrated ADCIRC response is greater than that Resio (2009) results, then a larger set of runs should be repeated to identify the PMSS.

6.) I believe that the adequacy of the applicant's analysis of the potential for breach of the cooling reservoir's containment due to of the embankment by currents and waves associated with storm surge is a moot point. Only the NRC SLOSH results indicate surge would reach the area of concern. I do not believe these are credible results. If additional runs using ADCIRC indicate inundation of the northern embankment, the velocity field generated by ADCIRC and the wave field generated by SWAN (or STWAVE) should be used to assess potential erosive forces and not the analyses presented in the SER.

I have also provided detailed recommendations in Sections **III-V** about the actions that may be taken to mitigate the concerns expressed in my comments.

III. SER Section 2.4S.5 – Probable Maximum Surge and Seiche Flooding

III.a. *2.4S.5.4.1 Probable Maximum Winds and Associated Meteorological Parameters*

The applicant determined the probable maximum meteorological winds (PMMWs) due to the probable maximum hurricane (PMH) using guidance in NWS Report 23 (NOAA, 1979). The applicant's PMMW parameters are presented in SER Table 2.4S.5-1.

SER Table 2.4S.5-1. Parameters of Probable Maximum Meteorological Winds

Parameter (units)	Symbol	Range of Values
Peripheral pressure (cm/in. of Hg)	P_w	76.50 / 30.12
Central pressure (cm/in. of Hg)	P_o	66.52 / 26.19
Pressure differential (cm/in. of Hg)	$P = P_w - P_o$	9.98 / 3.93
Radius of maximum winds (nautical miles)	R	5 to 21
Forward speed (knots)	T	6 to 20
Hg = mercury; in. of Hg = one-thirtieth of atmospheric pressure (e.g., 0.49 psia).		

Using these characteristics of the PMH, and guidance from NWS Report 23, the applicant estimates that the PMMW range for a stationary hurricane is 68.0 to 71.5 m/s (152 to 160 mph).

NRC staff issued RAI 02.04.05-6 inquiring whether any effort was made to adjust the estimated PMH parameters in light of more recent hurricanes that have occurred since the publication of the NWS 23 report. The applicant's response refers to a recent analysis by NOAA indicating that the period between 1945 and 1970 is considered a hurricane period that was as active as hurricane periods in the most recent decades. The applicant concludes that because the 1945 through 1970 period is covered by the analysis in the NWS 23 report, a PMH estimated using NWS 23 will provide a conservative assessment and will account for any increase in hurricane strength due to future climate variability.

NRC staff developed an independent estimate for the PMH / PMMW for the STP site also using NWS 23. Parameters are given in SER Table 2.4S.5-2.

SER Table 2.4S.5-2. NRC Staff's Estimates of PMH / PMMW Parameters

Parameter (units)	Value	Source in NWS 23
Latitude (degrees North)	28.6	
Coriolis parameter f (1/s)	7.1×10^{-5}	
Coastal distance (km / nautical mile)	601.9 / 325	Figures 1.1 and 1.2
Central pressure P_o (cm / in. Hg)	66.52 / 26.19	$P_w - \Delta P$
ΔP (cm / in. Hg)	9.98 / 3.93	Figure 2.3
Peripheral pressure P_w (cm / in. Hg)	76.5 / 30.12	Section 2.2.2
Radius of maximum winds R (mi)*	8-33.8 / 5-21	Figure 2.5
Forward speed T (m/s / knot)	3.1-10.3 / 6-20	Figure 2.7
Direction (degrees clockwise from North)	85-190	Figure 2.9
Coefficient K	79.5	Figure 2.11
Moving hurricane gradient vel. (m/s / mph)	70.5 / 157.6	Equation 2.2
Stationary hurricane gradient vel. (m/s / mph)	66.92 / 149.7	Equation 2.4

**REVIEWER'S NOTE – The radius of maximum winds in NWS 23 = 5-21 nautical miles, which is approximately 6-24 miles or 9–39 km. The units in SER Table 2.4S.5-2 appear to be incorrectly reported as miles and should be nautical miles. Also, it is not clear what the units are for the 8-33.8 values reported in this table.*

NRC staff-estimated stationary hurricane wind speed of 149.7 mph is consistent but slightly lower than the applicant's estimated range of 68.0 to 71.5 m/s (152 to 160 mph).

To compare the relative severity of the PMH parameters estimated from NWS 23, NRC staff compared them to other studies that have currently been carried out in the area (Resio 2009; Vickery 2009). Staff found that the PMH estimated from NWS 23 is smaller in size than those estimated near the STP site by Resio (2009), but it has greater wind speeds. On the other hand, the severe storms estimated by Vickery (2009) near the STP site are smaller in size than the PMH, but they have slightly greater wind speeds.

REVIEWER COMMENT 1: PMH parameters identified by both the applicant and NRC staff have upper limits of a category 4-5 intensity storm and a radius to maximum winds (RMW) of 24 statute miles. Given the landfall conditions of the strongest of the recent storms in the Gulf of Mexico (Katrina – cat 3, RMW=30 mi; Rita – cat 3, RMW=23 mi; Gustav – cat 2, RMW=27 mi; Ike – cat 2; RMW= 35 mi), I believe that the PMH radius to maximum winds is underestimated in NWS 23. This is especially true considering that in some cases these storms were significantly larger in size in the 1-2 days prior to landfall.

Storm surge is very sensitive to storm size (e.g., the storm surge due to Katrina – cat 3 and RMW=30 mi was larger than the storm surge due to Camille – cat 5 and RMW=15 mi even though both storms made landfall in a similar location) and therefore, it is critical that upper end RMW values are considered when determining the PMSS. It appears that only the storms used by Resio (2009) include larger values of RMW at landfall.

REVIEWER RECOMMENDATION 1: It is critical that large RMW values are considered when determining the PMSS and therefore the storm surge and inundation due to a PMH having an RMW of at least 35 statute miles should be evaluated. At a minimum, the PMSS due to a category 3-4 storm with large RMW should be compared with that already computed for a category 5 storm with the smaller RMW to determine the most conservative conditions. More recent statistical analysis of Gulf of Mexico hurricanes may provide additional guidance of the possible co-occurrence of extreme PMH parameters. The applicant should adopt an approach that more closely follows that of Resio (2009) in this regard.

REVIEWER COMMENT 2: There is an increasing body of evidence suggesting that storms will become stronger as the climate warms, although it is not clear how this will affect the entire suite of storm parameters, including RMW, that comprise a PMH. Despite this uncertainty, the applicant's conclusion that the PMH parameters obtained from NWS 23, "... will account for any increase in hurricane strength due to future climate variability" does not appear to be well justified.

REVIEWER RECOMMENDATION 2: Uncertainty in the relationship between climate warming and hurricane strength and the lack of information on other, co-occurring storm parameters makes the identification of future PMH parameter values problematic. Therefore, at a minimum, the most conservative PMSS computed using present day PMH parameters should be determined, e.g., using the larger storm size identified above. Sensitivity to increased storm strength and size (e.g., by 10-20 percent) should be evaluated to document the possible impact of climate warming on the PMSS at the STP site.

III.b. *2.4S.5.4.2 Surge and Seiche Water Levels*

The ABWR DCD Section 2.1 requires that the design-basis flood elevation shall be no greater than 0.3 m (1 ft) below site grade; the site grade is 10.4 m (34 ft) MSL.

The applicant's analysis of historical hurricane surge elevations indicates a peak storm surge elevation for a site close to STP Units 3 and 4 of approximately 4.9 m (16 ft) MSL.

The applicant used four different models to estimate potential storm surge flooding elevations near the STP site and the NRC staff performed independent confirming analyses using the SLOSH model. The individual SLOSH model runs are described in RAI 02.04.05-10 but are not included in the SER, presumably because the ADCIRC model results were assumed to be more reliable.

Static water level offsets were included to account for an "initial rise in water level" (=0.73 m = 2.4 ft - which is defined in RG 1.59 as a forerunner or sea level anomaly and might also be assumed to include steric expansion of the water column), a 10 percent exceedance of the astronomical high tide and sea level rise over the next 100 years. Values used in each modeling exercise are summarized in Table 1 in RAI 02.04.05-10.

1.) The SURGE model (Bodine, 1971). The applicant's analysis examines a range of values for wind and bottom frictions, PMH geometries, and track speeds. The maximum surge estimates at the coast of 6.1 m (20.04 ft) MSL include a sea-level rise of 0.59 m (1.93 ft) due to global climate change. To estimate the storm surge level near the STP site, the applicant uses this elevation as the downstream boundary condition for a backwater calculation of the Colorado River using the HEC-RAS model. The HEC-RAS model simulates the effect of a 100-year river flood event combined with the SURGE results. The resulting water surface elevation at the site is 7.4 m (24.29 ft) MSL.

   The NRC staff performed an independent analysis using the applicant's implementation of the SURGE model and confirmed the applicant's analysis. NRC staff did not perform independent analyses of the HEC-RAS results because their independent analysis of the PMH storm surge estimate using the SLOSH model was more conservative.

2.) Extrapolated results from archived SLOSH MOMs. SLOSH MOMs for Category 1 - 5 hurricanes were used to account for PMH conditions near the STP site. While, none of the archived SLOSH results indicates the inundation of the STP site, the most extreme of the storms in the archive is weaker than the PMH. Therefore, the applicant extrapolated the SLOSH results to estimate a PMH water surface elevation of 9.7 m (31.7ft) MSL – (reported as 31.1ft MSL in FSAR and in Table 1, RAI 02.04.05-10).

   NRC staff issued RAI 02.04.05-4 requesting the applicant to explain (1) how SLOSH Maximum of Maximum (MOM) water-level predictions were extrapolated to account for the PMH conditions; (2) whether the PMH used in this extrapolation was the same as the PMH used in the SURGE analysis to estimate the PMSS at the coast near Matagorda, Texas; and (3) how the applicant verified that the extrapolation is valid and conservative. The applicant response indicated:

   i. A third-order polynomial curve was used to fit between the NOAA pre-computed Categories 1 through 5 SLOSH MOM surge values against ΔP. The response provides the curve-fit procedure and describes its use. The staff verified the applicant's results using the PMH pressure differential.

   ii. There are differences in the PMH used with SURGE and the SLOSH MOM water levels in terms of the hurricane forward speed and the radius to the maximum winds. The applicant's assessment maintains that these differences are not important.

iii. The conservatism of the SLOSH extrapolation is based on the fact that the extrapolated value is larger than a similar assessment made using the SURGE model. NUREG–0933 "A Prioritization of Generic Safety Issues - Item C-14: Storm Surge Model for Coastal Sites (Rev. 1)," dated 2007 refers to SURGE as a conservative model.

NRC staff issued RAI 02.04.05-9 requesting the applicant provide a physical basis for the MOM–ΔP relationship with a citation to an accepted and validated method that uses such a relationship, or provide a justification with a citation indicating why estimating parameters of a third-degree polynomial relationship from five data points would result in an accurate estimation of the model parameter values. The applicant response cited NUREG–0933 that a bathystrophic model, SURGE, is adequate for calculating design-basis water levels.

The staff concluded that the applicant's extrapolation based on the MOM–ΔP relationship may not yield conservative estimates of peak water levels at the site, because there is no physical basis for choosing the extrapolation equation that the applicant uses. The staff independently estimated the PMH water surface elevations at the STP site using the SLOSH model and found that the surge simulated by the SLOSH model is higher than the applicant's initial SURGE model estimate.

3.) SLOSH model runs for PMH storms: The NRC staff carried out 162 SLOSH simulations using 81 storm parameters that represent the variability of PMH conditions at the STP site and 2 vertical datum shifts (see table).

NRC Staff SLOSH Run Parameters – 162 Total Combinations

Parameter (units)	Value
Coastal distance (km / nautical mile)	(1) distance equal to the radius of the maximum winds, west of the mouth of the Colorado River Navigation Channel at the barrier islands; (2) centered on the mouth of the Colorado River Navigation Channel at the barrier islands; (3) a distance equal to the radius of the maximum winds east of the mouth of the Colorado River Navigation Channel, at the barrier islands.
ΔP (cm / in. Hg)	???
Maximum Wind Speed	???
Radius of maximum winds R (km / mi)	9.7, 20.8, 33.5 km / (6, 12.9, 20.8 mi)
Forward speed T (m/s / knot)	3.1, 6.4, 9.8 m/s / (6, 12.5, 19 knots)
Direction (degrees clockwise from North)	97.5, 143.8, 190
Initial vertical offset from NGVD29 (m / ft)	1.1, 1.5 m / (3.5, 4.93 ft)

The two vertical offsets were computed as

- 1.1 m (3.5 ft) above NGVD29 for present day conditions
- 1.5 m (4.93 ft) above NGVD29 for 100 yrs of sea-level rise

The SLOSH simulations indicated that the maximum storm surge near the STP Units 3 and 4 site would be produced by a large (radius to maximum winds), fast-moving (forward speed) storm that would produce prevailing winds blowing from the east toward the STP Units 3 and 4 site (storm passing to the west of the site).

The version of the SLOSH model the NRC staff used truncated water surface elevations higher than 11 m (36 ft) NGVD29 and therefore the actual values of the storm surge water surface elevation were not retained. The staff's simulations resulted in the STP site being inundated during the most severe of the 81 PMH scenarios simulated; the storm surge water surface elevation on the grid cell where the STP Units 3 and 4 site is located exceeded 11 m (36 ft) NGVD29. Based on values of storm surge water surface elevations at surrounding grid cells, the staff estimated that the storm surge water surface elevation at the grid cell where the STP Units 3 and 4 site is located would probably be between 11.3 and 11.6 m (37 and 38 ft) NGVD29.

The corresponding water depth near the site would be approximately 0.9 to 1.2 m (3 to 4 ft) at this location. For this shallow water depth, the PMH wind speeds, and unlimited fetch, the staff estimated the wind-wave amplitude to be 0.27 to 0.36 m (0.9 to 1.2 ft). Based on the conservative assumption of an armored shore, the staff used a slope of 10 percent and determined a corresponding conservative wave runup to be approximately 0.20 m (0.65 ft). Therefore, the wave action adds 0.47 to 0.56 m (1.55 to 1.85 ft) to the peak level of inundation estimated by the SLOSH simulations.

The staff estimated the maximum PMH storm surge water surface elevation to be between approximately 11.8 to 12.2 m (38.6 to 39.6) NGVD29, including the effects of wind waves at the STP Units 3 and 4 site.

The staff compared the severity of their PMSS estimated for this study, to results from other concurrent studies (Resio 2009; Vickery 2009). The PMH estimated by the NWS 23 method is smaller in size than 20 parameter combinations reported for storm surge studies by Resio (2009), but it has greater landfalling wind speeds. Based on the copy of the Resio (2009) presentation provided to the reviewers, it is not clear how his parameter values were chosen. The storm surge estimated by ADCIRC(+STWAVE) for the 20 parameter combinations presented in Resio (2009) attains a maximum elevation of 30.4 ft near the STP site. These results clearly show that storms with slower translation velocities give greater surge at the STP site than those with faster translation velocities. On the other hand, the severe storms estimated by Vickery (2009) near the STP site are smaller in size than the PMH, but have slightly greater wind speeds (although by how much is not clear). Vickery (2009) carried out simulations of storm surge at the STP site using the basic SLOSH model. At the 100,000 year return period, the still water surface elevation is 24-25 feet in the vicinity of the STP site which is substantially less than that computed by the staff's independent SLOSH analysis described above.

4.) ADCIRC model runs for PMH storms: The applicant initially presented a single ADCIRC model run at the site audit Aug 31 - Sept 1, 2010. After the site audit, the staff issued Supplemental RAI 02.04.05-11 requesting the applicant to provide the following additional information:

i. a detailed description of the ADCIRC model, including the wind-wave submodel;

ii. a detailed description of supporting data sets, including the topographic and bathymetric grids;

iii. a list of conservatively selected plausible PMH scenarios consistent with the NWS 23 ranges of the PMH parameters used as inputs to the ADCIRC;

iv. a description and justification of why other plausible PMH scenarios were not selected as conservative;

v. a description of the sensitivity of the ADCIRC-simulated PMSS to the PMH parameters including the radius to maximum winds, forward speed, track direction, and the landfall location;

vi. a description of nonlinearity in the estimated PMSS corresponding to various combinations of PMH parameters;

vii. the selected PMSS near the STP site, including the wind-wave runup;

The applicant's response to RAI 02.04.05-11 includes the following.

i. A few notable parts to the ADCIRC description:

A spatially varying friction was used for low-velocity deeper offshore waters, shallow near-shore waters, rivers and inlets where velocities are expected to be higher, and in the remaining areas of the domain.

ADCIRC uses the USGS National Land Cover Data Classification map and land roughness lengths derived from the Federal Emergency Management Agency (FEMA) HAZUS software program to decrease overland winds due to land cover.

The applicant carried out an extensive validation of the ADCIRC predictions on the Texas coastline for historical hurricanes including Rita and Ike.

ADCIRC is tightly coupled to SWAN to estimate the wave setup.

FEMA has certified the ADCIRC for use in the development of Flood Insurance Rate Maps that need to account for flooding from storm surges. ADCIRC is the standard coastal model used by the USACE.

ii. The ADCIRC grid (Texas topographic grid version 12 or TX2008 [version 13 is identified in response to RAI 02.04.05-10]) is highly detailed from Brownsville to Port Arthur, Texas and extends inland to the 9- to 23-m (30- to 75-ft) elevation contour. Grid resolution is about 30 m (100 ft) in this region. It was initially built using the 10-m LIDAR data and later refined using the 1-m LIDAR data to include hydraulically relevant features such as levees, riverbanks and roads. The grid incorporates the Brazos, Nueces, and Rio Grande rivers and major dredged navigation canals such as the Gulf Intracoastal Waterway. The alignment of major topographic features including roads, shorelines, and rivers was checked against aerial photographs and satellite images. The bathymetry for inland waterways in coastal regions of Texas was derived from regional bathymetric and dredging surveys from the USACE, NOAA, TWDB, or nautical charts. The geometry, bathymetry, and topography represent post-Hurricane Ike conditions. Features higher than 3 m (10 ft) from the surrounding area (e.g. significant levee systems, elevated roads, and railroads) are incorporated as subgrid scale weirs or lines of nodes with crown elevations.

    Two of the features that the ADCIRC computational grid resolves with greater vertical accuracy than in the SLOSH computational basin for the Matagorda Bay area are the City of Matagorda levee and the dredge piles along the lower Colorado River. The City of Matagorda levee lies directly in the path of a hurricane storm surge as it advances from the open waters of the Gulf of Mexico toward the STP site. NRC staff concluded that these features of the ADCIRC bathymetric and near-shore topographic data provide more detailed site-specific information for storm surge simulation at the STP site compared to the SLOSH model

iii. Based on the NWS 23 analysis and the staff SLOSH runs, ADCIRC runs were made using a hurricane having a radius to maximum winds of 38.6 km (24 mi, 21 nm); an approach direction of 135 degrees clockwise from the north; a forward speed of 37 km/hr (23 mph, 20 kt); a central pressure of 887 mb (26.19 in. Hg); and a peripheral pressure of 1,020 mb (30.12 in. Hg). The only variables were the distance of the storm track from the site and the track's direction. The applicant used seven ADCIRC scenarios (summarized in SER Table 2.4S.5-3).

SER Table 2.4S.5-3. PMH Scenarios for ADCIRC Simulations

Scenario	Distance from Site	Track Direction	Maximum PMSS Water Surface Elevation
1	19.3 km (12 mi, 10.4 nm)	NW	8.1 m (26.5 ft) MSL
2	38.6 km (24 mi, 20.9 nm)	NW	8.9 m (29.3 ft) MSL
3	57.9 km (36 mi, 31.3 nm)	NW	8.7 m (28.5 ft) MSL
4	38.6 km (24 mi, 20.9 nm)	N	7.6 m (25 ft) MSL
5	38.6 km (24 mi, 20.9 nm)	N-NW	8.8 m (29 ft) MSL
6	38.6 km (24 mi, 20.9 nm)	W-NW	7.9 m (26 ft) MSL
7	38.6 km (24 mi, 20.9 nm)	W	6.1 m (20 ft) MSL

The initial conditions for the ADCIRC runs consisted of a water surface elevation that accounted for a 10 percent exceedance high tide, initial rise, and long-term sea-level rise estimated by NOAA.

The Bailey (2010) presentation indicates that the radial wind profile (i.e., the variation of wind speed with distance from the center of the storm) in the SLOSH wind model over estimates wind velocity outside the RMW as compared to other wind profile models. The applicant modified ADCIRC's wind model by replacing its Holland radial wind profile with the SLOSH radial wind profile, presumably to allow a closer comparison between ADCIRC and SLOSH and to generate a more conservative surge response in ADCIRC.

iv. The applicant selected PMH scenarios that represent the most conservative combination of storm scenarios, because the selected storm scenarios use the greatest ΔP that results in the strongest storm, the greatest radius to maximum winds that results in the largest storm, the greatest forward speed that increases surge heights, a maximum sustained wind speed that remains constant until landfall, tracks that are least resistant to wave build-up, and a conservative wind profile (the SLOSH wind model).

v. The maximum surge heights are listed in Table 2.4S.5-3. The greatest storm surge occurs when the storm passes the site at a distance equal to the radius of maximum winds and the storm track direction is generally to the northwest.

vi. To a limited degree, surge elevations do not vary linearly with track direction or distance from the site. The applicant also states that it was difficult to describe the nature of the nonlinearity, although the outcomes were consistent with the behavior of hurricane storm surges in the western Gulf of Mexico.

vii. Based on ADCIRC simulations using the SLOSH radial wind profile, the estimated PMSS at the STP Units 3 and 4 site is 8.9 m (29.3 ft) MSL including wave setup.

These ADCIRC model results provide the basis for the NRC staff's determination that the applicant has selected conservative PMH scenarios for estimating the PMSS at the STP site. NRC staff also determined that the applicant selected an appropriate model supported by site-specific information. Therefore, the staff concluded that the applicant's ADCIRC simulations for determining the PMSS at the STP site are adequate.

REVIEWER COMMENT 3: While the SURGE model may provide a quick sanity check on surge calculations in open coastal areas, it is lacking important physical processes that can contribute significantly to the surge. It clearly does not represent the current state of practice in this field and cannot be considered reliable in a predictive mode over land and perhaps not even in open coastal settings. The coupling with HEC-RAS to compute inundation and therefore water level at the STP site is also not reliable.

REVIEWER RECOMMENDATION 3: Results from the SURGE / HEC-RAS model combination should be discounted for the current study. In particular, the argument that other model results are sufficiently conservative because they generate higher water levels than SURGE / HEC-RAS, has no physical basis and should be rejected.

REVIEWER COMMENT 4: While extrapolation of SLOSH MOMs may provide a reasonable estimate of the corresponding SLOSH MOMs for the higher wind speed of the PMH, it does not address under estimates of the surge due to small PMH RMW values or fundamental errors in SLOSH due to poor grid resolution, inadequate domain size or specific model parameter values. Individual model results that isolate specific PMH parameters may be more helpful for determining PMSS than composited MOMs.

REVIEWER RECOMMENDATION 4: Results from the extrapolated SLOSH MOMs should be discounted for the current study. Errors and uncertainty in the basic SLOSH model setup and the compositing of results make the interpretation of these results unreliable. As noted above, the argument that these PMSS results are sufficiently conservative because they are higher than those computed using SURGE / HEC-RAS should be rejected.

REVIEWER COMMENT 5: NRC staff model runs using SLOSH confirmed that the greatest surge was generated by the largest radius storms, on a track that generated prevailing winds from the east and was located approximately 1 RMW to the west of the STP site. As noted above, it appears that the largest RMW may be an underestimate of the PMH and therefore the computed PMSS may also be an underestimate (depending on the assumed covariance between RMW and storm strength).

In addition, a recent evaluation of SLOSH for hurricane Ike has identified the small SLOSH domain (basin) size and possibly the frictional parameterization and lack of integrated wave forcing as generating under predictions of the Ike storm surge along the open coast by 25 – 33 percent (~1-2 m) compared to observations and ADCIRC model results. This is probably due to SLOSH's inability to capture the 2-3 m geostrophic setup due to the shore parallel winds that occurred during the day preceding Ike's landfall and that raised coastal and inland water levels prior to the arrival of the main surge generating winds. While RG 1.59 suggests a 0.75m "initial rise in water level", this is far below the Ike setup, particularly if the steric effect is included. Therefore, I am concerned that SLOSH may systematically under predict storm surge in this region given accurate wind forcing.

I am also concerned that SLOSH's inability to represent the geostrophic setup that precedes hurricane landfall coupled with the use of undersized storms, may have caused NRC staff to conclude that a fast approaching storm would cause the highest surge. In fact, a slower moving, large storm provides time for the generation of the geostrophic setup (that SLOSH misses altogether) and inland penetration and may therefore generate the PMSS. This concern is consistent with the results from Resio (2009) which showed that greater surge is caused by storms with slower forward speeds. Unfortunately, as noted below, the applicant's ADCIRC runs utilized the NRC's preliminary SLOSH findings and only considered short duration, fast moving storms.

Bailey (2010) indicated that SLOSH has relatively coarse grid resolution in inland areas and this is likely to compromise the accuracy of the results.

Vickery's (2009) SLOSH model runs gave 100,000 year surge elevations near the STP site that were approximately 24-25 ft, which is 13 ft less than NRC staff. It is not clear why such a substantial difference exists between the Vickery and NRC results.

Since ongoing studies are showing that SLOSH systematically misses several important aspects of storm surge, the historically quoted SLOSH skill statistics may be more a reflection of storm specific calibration than predictive ability.

REVIEWER RECOMMENDATION 5: The NRC staff's SLOSH runs confirm the role of larger storms, the track location and direction in generating the PMSS. However, there are a number of serious issues with the implementation and use of SLOSH at this location that make the computed PMSS values and the conclusion that storms with greater forward speeds give more surge both unreliable.

REVIEWER COMMENT 6: The application of ADCIRC to any geographical region requires the setup of an unstructured model grid and the establishment of spatially varying model parameters (including friction coefficients, roughness and canopy factors, a continuity equation weighting factor, etc). Much progress has been made towards automating this progress based entirely on supporting data sets (e.g., LIDAR topography, NLCD land use data), however, judgment remains integral to the process and therefore the experience and skill of the group applying ADCIRC certainly play a role in the reliability of the results. Based on the original and supplementary materials provided for review, it appears that this has been done in a reasonable manner.

The ADCIRC results presented in the STP PMSS study leverage a National Flood Insurance Program study that has been ongoing for the state of Texas for a couple of years. In all such FEMA studies, model validation is required to demonstrate that the model has been capably applied and that the results are reliable in the area of interest. It seems reasonable that the STP study should require validation documentation of the specific implementation of ADCIRC that is used to determine the PMSS. Unfortunately, while validation is alluded to in the SER and in the applicant's response to specific reviewer requests to see such validation, it was not provided for the STP study.

As discussed previously, I believe that the PMH RMW used in the ADCIRC analysis is too small and that this may have caused an underestimation of the PMSS at the STP site. Use of the SLOSH results to select only fast moving storms, coupled with the short offshore duration of the ADCIRC runs (see below) may also have led to inadequate time to develop the pre-storm setup and inland penetration and therefore caused the PMSS at the STP site to be underestimated.

My inspection of the modified wind model code used in the ADCIRC runs indicates that the Holland radial wind profile was replaced by the SLOSH radial wind profile. However, this does not mean that ADCIRC and SLOSH generate the same wind velocities. For example, ADCIRC converts its winds from an assumed 1 min averaging time interval to a 10 min averaging interval (by multiplying by a coefficient of 0.88) before using them to compute the wind forcing. I do not know whether SLOSH does this as well. Other subtle differences between the wind forcing computed by ADCIRC and by SLOSH include how the translation velocity is dealt with (ADCIRC assumes it is part of the specified Vmax some models do not), the assumed density of air and the surface drag law (some of these issues are discussed in the presentation by Vickery 2009). Thus it is likely that the magnitude of the ADCIRC winds and the resulting ADCIRC wind forcing (proportional to the wind speed squared) are less than SLOSH, even though the SLOSH radial wind profile is used in both models. Recent experience modeling hurricane Ike indicates that ADCIRC generates larger surge than SLOSH when truly identical wind forcing is utilized. The finding that SLOSH gave larger surge levels than ADCIRC as the STP site further suggests the ADCIRC wind velocities used in this study were actually smaller than the SLOSH winds velocities. At a bigger picture level, it is not clear what the value is of using the SLOSH wind model over the Holland wind model that is available in ADCIRC. The ADCIRC / Holland wind model allows the specification of more parameter values than the SLOSH model, thereby providing the opportunity to create a wind field that is more consistent with the selected PMH parameter values and therefore should be more useful for the STP PMSS analysis.

Given the large ADCIRC domain size, it should be possible for ADCIRC to explicitly model the "initial raise in water level" (with the exception of the steric adjustment) that is added into the other model results. Therefore, it would seem that this factor should be decreased for use in ADCIRC model runs.

REVIEWER RECOMMENDATION 6: Explicit validation of the version of ADCIRC used in the current study should be provided in the vicinity of the STP site. Additional ADCIRC model runs, beyond those presented by the applicant, are needed to evaluate the PMSS at the STP site. Differences between SLOSH and ADCIRC are expected due to differences in the models' setups as well as likely differences in the wind forcing. The Holland radial wind profile in ADCIRC or a more sophisticated wind model should be used to make any additional ADCIRC PMSS runs at the STP site (and not the modified wind model that uses the SLOSH radial wind profile).

REVIEWER COMMENT 7: Resio (2009) presents what appears to be a systematic study of storm surge in the vicinity of the STP site that is more thorough than those conducted by the applicant and the NRC staff. While the powerpoint presentation of this study provided for review was short of details and context, it indicates a PMSS value of approximately 30 ft in the vicinity of the STP site. In the fall of 2009 – spring 2010, ADCIRC was substantially recalibrated in the Gulf of Mexico based on an extensive storm surge data set collected for hurricane Ike. Resio's study was done before this re-calibration and it is not clear how this would affect the ADCIRC results at the STP site.

REVIEWER RECOMMENDATION 7: It appears that Resio (2009) is the most credible study done to identify the PMSS at the STP site. I recommend repeating one or two of the runs in this study that resulted in the greatest surge levels near the STP site using the re-calibrated ADCIRC. If the results are comparable with Resio's original results, it is highly likely that these accurately represent the PMSS at the site.

III.c. *2.4S.5.4.3 Wave Action*

The PMSS elevations obtained from the SLOSH analysis performed by NRC staff yield a corresponding still water depth near the site of approximately 0.9 to 1.2 m (3 to 4 ft). For this shallow water depth, the PMH wind speeds, and unlimited fetch, NRC staff estimated the wind-wave amplitude to be 0.27 to 0.36 m (0.9 to 1.2 ft). Based on the conservative assumption of an armored shore, the staff used a slope of 10 percent to determine a conservative wave runup of approximately 0.20 m (0.65 ft). Wave action was determined to add 0.47 to 0.56 m (1.55 to 1.85 ft) to the peak level of inundation estimated by the SLOSH simulations.

Therefore, the staff estimated the maximum PMH storm surge water surface elevation to be between approximately 11.8 to 12.2 m (38.6 to 39.6 ft) NGVD29, including the effects of wind waves at the STP Units 3 and 4 site.

ADCIRC is tightly couple with the SWAN wave model and therefore the surge includes wave induced setup (which none of the other models include). Since the PMSS water surface elevation of 8.9 m (29.3 ft) MSL is below the grade surrounding the STP site, wave action coupled with the PMSS is not the controlling wave scenario. The applicant concluded that wave action associated with dam breach flooding would be more conservative than that associated with storm surge.

NRC staff determined that the applicant's site-specific PMSS maximum water surface elevation of 8.9 m (29.3 ft) MSL is reasonable and conservative. Although the applicant does not provide an estimate of the wind-wave runup, the staff determined that the applicant's independent estimate of 0.20 m (0.65 ft) could be conservatively added to the applicant's PMSS stillwater and wind setup estimate, because the staff's estimate is derived from a more conservative PMSS scenario. Therefore, the staff concluded that the maximum PMSS water surface elevation at the STP Units 3 and 4 site accounting for the wind setup and runup would not exceed 9.1 m (30 ft) MSL and would be 0.6 to 1.2 m (2 to 4 ft) below the STP Units 3 and 4 site grade of 10.4 to 11 m (34 to 36 ft) MSL. Because the PMSS maximum water surface elevation accounting for wind-wave effects is below the site grade and is exceeded by the maximum water surface elevation expected during the postulated main cooling reservoir embankment breach event, the staff concluded that further investigation of the PMSS at the STP site is not warranted.

REVIEWER COMMENT 8: While the above analysis may be reasonable, ADCIRC coupled to SWAN (or STWAVE) will provide estimates of wave properties throughout the region of inundation.

REVIEWER RECOMMENDATION 8: Make use of the wave properties directly from ADCIRC + SWAN (or ADCIRC + STWAVE) to assess wave action at the STP site if additional model runs indicate the area is inundated.

III.d. *2.4S.10 Flooding Protection Requirements*

RAI 02.04.05-11 requested additional information regarding the PMSS estimation at the STP site and a possible failure of the main cooling reservoir northern embankment due to erosive action of PMSS waters. RAI 02.04.05-11 items (i) – (vii) are summarized in section **III.b**. above. The following items are relevant to the current section:

viii. a detailed description of various methods used to estimate current velocities during a PMSS event;

ix. a detailed description and justification of the simplifying assumptions;

x. conservatively selected current velocities and the durations that these currents will affect the main cooling reservoir embankment; and

xi. relevant citations to support a justification for the ability of the grass-lined outer face of the northern main cooling reservoir embankment to withstand the current velocities without erosion severe enough to cause an embankment breach.

The applicant provided the following responses to these items:

viii. The outer face of the main cooling reservoir northern embankment is grass-lined with a slope of 3 horizontal to 1 vertical.

The ADCIRC prediction of the PMSS water surface elevation at the STP site is 8.9 m (29.3 ft) MSL, which is lower than the grade elevation of 10.4 m (34 ft) MSL at the northern face of the main cooling reservoir northern embankment.

The applicant's SLOSH prediction of the PMSS water surface elevation at the STP site, is 11.7 m (38.5 ft) MSL and the coincident wind-wave action would raise the storm surge water surface elevation to 12.7 m (41.8 ft) MSL. The time history of this very conservative scenario showed that the PMSS water surface elevation would be at 10.4 m (34 ft) MSL (i.e., at site grade) for 80 minutes; at or above 11 m (36 ft) MSL for 50 minutes; and at or above 11.6 m (38 ft) MSL for 25 minutes. The applicant states that significant erosion of the grass-lined north face of the main cooling reservoir northern embankment would not occur during this short amount of time, because a grass surface works well for short-term exposure as plant roots keep soil particles bound together to create a flexible system that deforms without tearing. The flood-protection levee for Texas City survived a sustained surge and wave attack during Hurricane Ike for many hours without a breach (USACE, 2009). The main cooling reservoir embankment is similar to but much larger than typical hurricane surge-protection levees that have mostly withstood major hurricanes in the past.

ix. A storm surge that would exceed the STP Units 3 and 4 site grade elevation of 10.4 m (34 ft) MSL is not a credible event. The applicant notes that ADCIRC predictions resulted in a PMSS water surface elevation of 8.9 m (29.3 ft) MSL, which is significantly less than the STP Units 3 and 4 site grade elevation. The applicant also states that even the very conservative predictions from the SLOSH resulted in a PMSS that would inundate only a small portion of the main cooling reservoir northern embankment for a short duration. The applicant concludes that any erosion at the base of the main cooling reservoir northern embankment would not threaten a failure.

x. The applicant states that water will flow past the main cooling reservoir northern embankment under the very conservative PMSS scenario predicted by the SLOSH. SLOSH does not output current velocities, but they can be estimated using (1) the area around the STP Units 3 and 4 that experiences the PMSS and matching the volume of water that fills and drains through this area during the PMSS event; (2) using Manning's n and a friction slope estimated by change in water surface elevations; and (3) tracking the PMSS wave-front past the site. The applicant uses all three methods to estimate current velocities during the PMSS event. The resulting maximum current velocities are 3.5 m/s (11.6 fps), 0.9 m/s (3.1 fps), and 1.9 to 4 m/s (6.2 to 13.2 fps), respectively. The PMSS flow past the main cooling reservoir northern embankment would occur for a maximum duration of 80 minutes.

xi. The USACE recommends a design velocity of 1.5 to 2.4 m/s (5 to 8 fps) for stable grass-lined flood channels. The applicant states that the grass-lined main cooling reservoir embankment can be expected to sustain a short exposure to current velocities slightly higher than those assumed in the design of flood channels that have a design life of several decades and would likely be subject to flow durations considerably longer than 80 minutes. The applicant concludes that erosion of the main cooling reservoir northern embankment is unlikely.

The NRC staff agreed with the applicant's conclusion that the maximum PMSS would not exceed 8.9 m (29.3 ft) MSL and accounting for wind runup would not exceed 9.1 m (30 ft) MSL. These elevations are 1.2 to 1.8 m (4 to 6 ft) below the STP Units 3 and 4 site grade of 10.4 to 11 m (34 to 36 ft) MSL.

The applicant states in FSAR Subsections 2.4S.4.2.2.2.2 and 2.4S.4.2.2.2.4.1 that the terrain immediately downstream of a service road running along the toe of the exterior slope of the main cooling reservoir northern embankment acts as a control against the development of a breach. The terrain elevation at this location is 8.8 m (29 ft) MSL. Because the maximum PMSS effects is 9.1 m (30 ft) MSL, NRC staff concluded that the lower reach of the toe of the main cooling reservoir northern embankment would experience currents during the PMSS event. The slope of the main cooling reservoir northern embankment at this location is 6 horizontal to 1 vertical. Because of the gentle slope and relatively small area of the toe of the main cooling reservoir northern embankment that would be inundated during the PMSS event, the staff concluded that it is unlikely that PMSS currents would cause significant damage to the toe of the northern embankment. For the main cooling reservoir embankment to fail, the erosive action of the PMSS current would have to erode the toe to such an extent that (1) a pipe would form extending to the interior face of the embankment; or (2) an extensive sliding surface would form extending from the downstream to near the upstream face of the embankment. Because the toe of the main cooling reservoir northern embankment is inundated only with a depth of 0.3 m (1 ft) near the exterior end of the embankment, the staff concluded that such a failure mechanism is unlikely.

Even in the unlikely scenario that the main cooling reservoir northern embankment were to fail because of erosive action of PMSS currents, the resulting flood would be similar to and not more severe than that reviewed by the staff in Section 2.4S.4 of this SER.

REVIEWER COMMENT 9: This issue seems rather academic, because only the NRC SLOSH results indicate surge would reach the area of concern. I do not believe these are credible results, making any analysis of the erosion of the main cooling reservoir's embankment a moot point. Thus, I have no specific comment regarding the subsequent analysis of the adequacy of the exterior of the cooling reservoir northern embankment to withstand hypothetical erosive forces due to local inundation and wave action.

REVIEWER RECOMMENDATION 9: If additional runs are performed using the ADCIRC model and these runs indicate inundation of the northern embankment, the velocity field generated by ADCIRC and the wave field generated by SWAN (or STWAVE) should be evaluated during the time this area is inundated to assess potential erosive forces.

IV. Response to RAI 02.04.05-10, Question 1

Much of the material covered in this response is included in the SER and is therefore reviewed in section **III**. The following presents a brief summary of relevant additional material that was not discussed in section **III**.

Based on its independent estimation of PMSS water surface elevations using SLOSH, NRC staff determined that the PMSS would be above the elevation of the toe of the cooling reservoir northern embankment and therefore it may be subject to wave action associated with the PMSS. Because the outer face of the MCR embankment is only grass-lined and not protected by reinforced soil-cement or riprap, the staff postulated that the MCR embankment could possibly fail during the PMSS event. If this scenario were to occur, the MCR breach flood would coincide with the PMSS event.

In this RAI, NRC staff asked for:

i. an analysis of the PMSS event using a conservative approach such as those predicted by a storm surge model (e.g., SLOSH) with input from appropriate PMH models.

ii. reasons why exposure of the outer face of the MCR embankment to the PMSS event would not lead to a breach, and

iii. if an MCR breach is postulated under PMSS conditions, a revised estimate of the design-basis flood water surface elevation at the STP site.

The applicant's response to RAI 02.04.05-10 includes the following.

i. The applicant has completed additional modeling of the PMSS using the latest versions of SLOSH and ADCIRC and the latest available digital elevation maps generated using LiDAR. There are now six separate PMSS predictions, including the SLOSH analysis performed by the NRC staff. These six predictions of PMSS are based on at least three separate storm surge computer models (i.e., SURGE, several versions of SLOSH, and ADCIRC) that predict the PMSS at the STP site. The results and the specific assumptions for-each model are listed in response to RAI 02.04.05-10 Table 1.

   The applicant concluded that among these six models (four of which were reviewed in the SER), ADCIRC is the most credible. Model validation for use in Texas in particular (including the STP vicinity), and in the Gulf of Mexico region in general, is more robust for ADCIRC than for SLOSH. As a result, ADCIRC is more widely used by both the USACE and FEMA to obtain detailed predictions of storm surge levels in these areas. The ADCIRC model is currently being used by the USACE for the design of flood and storm damage reduction projects and infrastructure all along in the Gulf Coast area, including Matagorda Bay, which is located in close proximity to the STP site. ADCIRC is currently being used by FEMA for the development of Flood Insurance Rate Maps for the area around the STP site.

   All of the models use initial conditions and assumptions that are consistent with recommendations in SRP 2.4.5 and RG 1.59 including the use of an initial water level rise, co-occurrence with a 10 percent exceedence high tide and a long term sea level rise (see Table 1). Each model utilizes winds developed from the PMH determined from NWS 23.

   The applicant's analysis is based on ADCIRC Version 49 and Texas Grid version 13 (version 12 is listed in the SER), utilizes ADCIRC's built in Holland wind model and assumes that PMH intensity starts to decay after the storm makes landfall. This model predicted the PMSS at the Gulf Coast is 21.5 feet MSL and the PMSS at the STP site is 26.5 ft MSL, which is less than the site grade of 34 ft.

ii. The applicant evaluated the potential that a PMSS could cause a breach of the north face of the MCR embankment based on the PMSS and wave runup predicted by Model 4 (SLOSH), which resulted in the highest prediction for PMSS, 38.46 feet MSL still water level, and 41.76 feet MSL with wave runup.

The north face of the MCR embankment was evaluated because of the proximity to safety-related structures and because a MCR breach at this location results in the highest flood levels near the power block.

Except for a wind direction from the southeast, the MCR would block waves or limit the fetch during the PMH so that significant wave generation would not occur near the north MCR embankment. With winds from the east when the surge elevation first reaches the site, waves would affect the eastern side of the MCR but there would be no direct wave impact on the northern portion of the levee. As the wind direction shifts from east to southeast and then south, the surge rises and then falls to below the site elevation. At no time during the PMSS passage do winds have a northerly component that could produce waves that would strike the northern side of the embankment.

While the postulated PMSS would not have winds or waves from the north, it would be possible to have a hurricane that did include winds from the north. For example, if a storm were to pass to the east of the facility it would produce northerly winds. However, such a storm would generate surge elevations much lower than the PMH. The PMH must pass to the west of the site to create the surge levels predicted by Model 4.

While the north portion of the embankment would not be exposed to wind-generated waves during the PMSS, it could experience strong currents along the outside of the levee, as a large quantity of water is moved past the site in a short period. The MCR levee is designed to contain water above ground level and the external side of the levee is a grass and maintained slope that is similar to levees designed specifically for protection from both hurricane surge and flooding rivers. A grass surface works well for short-term exposure because plant roots act to keep particles of soil together, creating a flexible system that can deform without tearing. Waves and currents of short duration (i.e., less than several hours) on a well-vegetated cohesive material embankment would not be expected to lead to erosion related concerns. Model 4, the worst case, predicts the surge level is at- or above 36 feet MSL for approximately 50 minutes. Expected erosion velocities are further clarified in the response to RAI 02.04.05-11 and the SER. Any erosion at the base of the levee that might occur with less than an hour of exposure to the current would not threaten the levee.

iii. The NRC staff believes that collectively the 6 models described in this response provide a very high degree of assurance that STP 3 & 4 structures, systems and components important to safety are not vulnerable to a PMSS based on their use of a design basis flood level of 40.0 feet MSL.

REVIEWER COMMENT 10: It appears that the initial implementation of ADCIRC is discussed in more detail in the FSAR 2.4.5 "Probable Maximum Surge and Seiche Flooding" presentation given at the NRC Audit (Bailey 2010). This single model run was superseded by the model runs done in response to RAI 02.04.05-11.

REVIEWER COMMENT 11: It appears that the northern embankment of the main cooling pond reservoir should not be vulnerable to significant wave damage associated with the PMSS due to the fact that surge inducing winds would not come from a northerly direction.

REVIEWER COMMENT 12: The highest PMSS values obtained at the STP site resulted from SLOSH model runs conducted by the applicant (Models 3, 4) and the NRC staff (Model 6).

V. Response to RAI 02.04.05-11

Much of the material covered in this response is included in the SER and is therefore reviewed in section **III**. The following presents a brief summary of relevant additional material that was not discussed in section **III**.

The response to RAI 02.04.05-10 and presentations by STPNOC during a site audit conducted by the NRC staff on August 31-September 1, 2010, provided detailed justification for the conclusion that the ADCIRC model provided the most reliable PMSS predictions for the STP site. ADCIRC predicted the PMSS for the STP site, including wave runup, is 26.5 feet MSL, which is significantly lower than the 34-foot MSL nominal plant grade at STP 3 & 4. In response to this RAI question, STPNOC performed additional ADCIRC modeling using "conservatively selected plausible PMH scenarios, consistent with NWS 23 ranges of PMH parameters." Using these very conservative assumptions which are detailed in this response, ADCIRC predicts the PMSS for the STP site, including wave runup, is 29.3 feet MSL, which is still significantly lower than the 34 foot MSL nominal plant grade at STP 3 & 4.

Items i - vii in this response provide the detailed description of the ADCIRC computer model and the assumptions used in the model that ensure that the results are based on "conservatively selected plausible PMH scenarios, consistent with NWS 23 ranges of PMH parameters." Using these assumptions, ADCIRC predicts the PMSS for the STP site is 29.3 feet MSL, including wave runup, which is significantly lower than the 34-foot MSL nominal plant grade at STP 3 & 4.

In this RAI, NRC staff asked for:

i. a detailed description of the ADCIRC model, including the wind-wave submodel;

ii. a detailed description of supporting data sets, including the topographic and bathymetric grids;

iii. a list of conservatively selected plausible PMH scenarios consistent with the NWS 23 ranges of the PMH parameters used as inputs to the ADCIRC;

iv. a description and justification of why other plausible PMH scenarios were not selected as conservative;

v. a description of the sensitivity of the ADCIRC-simulated PMSS to the PMH parameters including the radius to maximum winds, forward speed, track direction, and the landfall location;

vi. a description of nonlinearity in the estimated PMSS corresponding to various combinations of PMH parameters;

vii. the selected PMSS near the STP site, including the wind-wave runup;

Information contained in the applicant's response to RAI 02.04.05-11 that was not explicitly covered in the review of the SER includes the following.

i. A description of ADCIRC's wind model based on Holland (1980). However, in the model runs completed for the response to this RAI, ADCIRC's Holland radial wind profile was replaced by SLOSH's radial wind profile.

   The ADCIRC model as applied to the STP analysis underwent an extensive flood level evaluation process to validate it over a range of conditions to ensure that the flow physics of the system were accurately characterized. The set of validation storms specific to the Texas coastal areas included Hurricanes Carla (1961), Celia (1970), Allen (1980), Alicia (1983), Bret (1999), Rita (2005), and Ike (2008). Hurricanes Rita and Ike were particularly useful storms for validation because of the large degree of surge they produced, and the accurate measurements of wind, atmospheric pressure, waves, and surge levels that exist for these two storms.

   Hurricane waves and storm surge as estimated by the coupled SWAN + ADCIRC model have been validated for Hurricane Katrina and Hurricane Rita, demonstrating the importance of inclusion of the wave-circulation interactions.

ii. The TX2008 mesh is refined locally to resolve features such as inlets, rivers, navigation channels, levee systems, and local topography/bathymetry. Previous mesh-resolution sensitivity studies applying the ADCIRC model to the rivers and the Lake Pontchartrain- Lake Borgne inlet system indicate that under-resolution severely dampens tidal and surge propagation into rivers and inlets.

   Regardless of channel dimensions, a small number of meshing stipulations were adhered to while mapping inland waterway bathymetry in the model. The most stringent constraint was to set a maximum resolution of 100 feet throughout the mesh in order to control computational cost. A finer level of resolution creates additional nodes, elements, and thus calculations per time step. In addition, a smaller time step is needed within the ADCIRC model in order to accommodate for the high spatial resolution. A second important attribute of channel meshes is the placement of a minimum number of nodes across a channel. When possible, at least five nodes were placed across a channel for two reasons. First and foremost, channels require high resolution in order to adequately capture bathymetric characteristics. Second, multiple nodes are placed within the channel to prevent the ADCIRC wetting and drying algorithm from artificially reducing the conveyance of the channel.

The TX2008 computational mesh contains more than 2.8 million nodes and nodal spacing varies significantly throughout the mesh. Grid resolution varies from approximately 12 to 15 miles in the deep Atlantic Ocean to about 100 feet in Texas.

Features were only included if the crown height was more than three feet above the adjacent topography and the feature was long enough to substantially impede flow. Raised features lying three meters or less above prevailing ground were defined using a continuous line of nodes to impede the flow. This definition allows the ADCIRC model to predict flow over relatively low-lying raised features dynamically using the shallow water equations. For these roads, railroads, and ridges, a line of ADCIRC nodes were carefully placed along the centerline of the feature. The finite element edges were aligned to follow the feature, thereby accurately implementing the topographic feature into the mesh. In instances that raised features are substantially higher than prevailing ground, it is most effective to treat these structures as sub-grid scale parameterized weirs within the domain. Sub-grid scale weirs were included with sub-and super-critical weir coefficients for features that were notably higher (i.e., 10 feet) than prevailing ground.

iii. The results of storm surge simulations using SLOSH indicated that the maximum water, surface elevation near STP Units 3 and 4 sites would be produced by a large (in terms of radius to maximum winds), fast-moving (in terms of forward speed) storm that would produce prevailing winds blowing from the east toward STP Units 3 and 4. Because hurricanes rotate counter clockwise in the northern hemisphere, the highest surges are expected on the east side of the hurricane eye due to the fastest onshore wind being toward the right of the eye. Storms with larger forward speeds generate faster responses in surge, leaving less time for the surge to dissipate over and around the surrounding terrain. Considering these factors, the site would be most vulnerable to flooding when the eye of the hurricane passes quickly to the west of the site on the leading edge of the storm. Based on the above outcomes and observations, STP concluded that the PMH estimated from the NWS 23 method would result from a storm with a radius to maximum winds of 20.8 miles, an approach angle of 143.8 deg clockwise from the north, and a forward speed of 21.8 mph.

Based on these results, a series of hurricane scenarios were simulated using ADCIRC to determine the maximum water surface elevation near STP Units

3 and 4 resulting from storm surge. The PMH parameters selected for the ADCIRC runs were based on the storm scenario that produced the maximum surge at the site during the prior analysis with SLOSH. Specifically, the PMH parameters selected for the ADCIRC runs based on NWS 23 are a radius to maximum winds of 24 miles (21 nm); an approach direction of 135 deg clockwise from the north (i.e. a northwesterly direction); a forward speed of 23 mph (20 knots); a central pressure of 26.19 in Hg; and a peripheral pressure of 30.12 in Hg. The only variables were the distance of the storm track from the site and the track direction. The resulting features of each of the seven simulated hurricanes are presented in **SER Table 2.4S.5-3**.

As discussed during the on-site audit by the NRC audit team on August 31 through September 1, 2010, when using similar storm features ADCIRC will produce smaller values for surge heights at the site of STP Units 3 and 4 when compared to values produced by SLOSH. The reasons for these differences are as follows:

- Grid Resolution
- Terrain Features (e.g., City of Matagorda Levee)
- Wind Model
- Friction Coefficients (Bottom and Surface)
- Pressure Differential (SLOSH: 133 Mb; ADCIRC: 123 Mb to 126 Mb)

When executing the ADCIRC runs, the program uses the Holland wind profile as the basis for calculating surface wind speeds as a function of distance from the storm center. SLOSH uses a different wind profile based on NWS 48. When comparing the two models, for the same gradient wind speed and distance from the storm center, the SLOSH wind profile based on NWS 48 will generate a greater value for wind speed than the Holland wind profile. The ADCIRC code was therefore changed, with the NWS 48 wind profile equation inserted in place of the Holland wind profile equation. The pressure differential was also increased during the ADCIRC runs to 133 Mb. These steps were taken to mitigate differences between the program outputs that might otherwise be attributable to the wind model and to the pressure differential. The only remaining differences between the two models are gird resolution, terrain features, and friction coefficients.

iv. The basis for selecting the storm scenarios as described in response to (iii) reflect ranges of PMH parameters for the STP site as set forth by NWS 23. The storm features represent the most conservative combination of storm scenarios in terms of features that contribute most to amplification of storm surge height. Specifically, the storms scenarios use:

- the greatest pressure differential which will create a stronger storm,
- the greatest radius to maximum winds which will create a larger storm,
- the greatest forward speed which will increase surge heights,

- a maximum sustained wind speed that remains constant up to landfall,
- tracks which direct storm surge in paths that are least resistant to wave build-up, and
- a wind profile that is considered conservative when compared to other wind profiles.

A credible storm scenario could not be identified within the parameters set forth in NWS 23 that would be viewed as more conservative than the seven scenarios as defined in Item iii.

v. ADCIRC was run for the seven scenarios as discussed in response to Item iii. Overflow errors were encountered within ADCIRC during execution of Scenario 7 with the storm heading in a westerly direction. Such errors occur when the grid cells become overwhelmed in both number and magnitude of surge height, thus resulting in termination of the program (see Scenario 7). When the program terminated, the storm had made landfall along the Texas coastline, with the STP site shown as dry. To determine if the STP site would have been inundated had the storm continued to progress inland following landfall, the scenario was re-run at a lower maximum wind speed of 140 knots versus 160 knots. Execution was subsequently successful, with the site showing a surge level of 18 feet above MSL. Earlier sensitivity runs using ADCIRC showed that a 20-knot difference in the maximum wind speed for a northwesterly track would produce a difference in surge height of approximately 1.5 feet. The estimated surge height for Scenario 7 as shown in Table 2 and Figure 5 was therefore estimated to be approximately 20 feet above MSL.

vi. The results as shown in response to Item v indicate, to a limiting degree, that the surge height does not vary linearly with respect to either track direction or distance from the site. It is difficult to describe the exact nature of this non-linearity, as it could be attributable to many factors. However, the outcomes are consistent with the observed behavior of hurricanes with respect to storm surge impacts along the western portion of the Gulf of Mexico.

vii. Based on the outcome as presented in response to Item v, the PMSS as generated by ADCIRC, using NWS 48 wind profile, is estimated to be 29.3 ft above MSL. This PMSS will occur as the result of a hurricane traveling in a northwesterly direction (i.e., an approach direction of 135 deg clockwise from the north) passing within 24 miles of the STP site. During its life up to the point of landfall, the storm will have a constant forward speed of 23 mph, a central barometric pressure of 887 Mb, and a maximum sustained wind speed of 160 knots (184 mph). Upon-landfall, the' storm will continue in a northwesterly direction and begin to decay gradually as it moves inland.

The resulting storm surge value of 29.3 feet is 2.8 feet greater (approximately 10 percent) than the value of 26.5 feet above MSL as presented to the NRC

Audit Team on August 31, 2010. The 26.5 ft value was estimated running ADCIRC with a Holland wind profile and a smaller pressure differential (126 Mb vs. 133 Mb).

Items viii - xi in the response to RAI 02.04.05-11 provide additional information about the role of the PMSS in causing a breach of the Main Cooling Reservoir Embankment. These items have been adequately reviewed in the earlier reviews provided for the SER and the response to RAI 02.04.05-10 and therefore no further review is provided.

REVIEWER COMMENT 13: The discussion presented in item iii concerning the reasons why the surge computed by SLOSH would be greater than that compute by ADCIRC may not be accurate. In particular, I disagree with the following statements in this section,

- "storms with larger forward speeds generate faster responses in surge, leaving less time for the surge to dissipate over and around the surrounding terrain"
- "As discussed during the on-site audit by the NRC audit team on August 31 through September 1, 2010, when using similar storm features ADCIRC will produce smaller values for surge heights at the site of STP Units 3 and 4 when compared to values produced by SLOSH. The reasons for these differences are as follows:
    - Grid Resolution
    - Terrain Features (e.g., City of Matagorda Levee)
    - Wind Model
    - Friction Coefficients (Bottom and Surface)
    - Pressure Differential (SLOSH: 133 Mb; ADCIRC: 123 Mb to 126 Mb)"

Recent careful comparisons between the surge computed for hurricane Ike using ADCIRC and using SLOSH indicates that ADCIRC generates approximately 25-33 percent greater surge than SLOSH when truly identical wind forcing is used. While the various reasons cited in the text may cause SLOSH to generate greater surge than ADCIRC, there is no *a prior* reason that this will be the case, particularly if the same pressure differential and wind model are used. Note, as discussed above, use of the SLOSH radial wind profile in ADCIRC does not mean the two models were operating with the same winds speeds or wind forcing.

REVIEWER RECOMMENDATION 13: The assertion that when using similar storm features ADCIRC will produce less storm surge than SLOSH should be discounted without further confirmation.

REVIEWER COMMENT 14: The storm parameters described in item iv may not be conservative because a larger radius storm was not considered. Furthermore, ADCIRC runs were initiated only 12 hrs before landfall. During these 12 hours the storm was ramped from category 1 to category 5. After making landfall the storm strength was held constant in time and in some cases diminished as it moved further inland. This storm representation coupled with the use of a fast forward speed greatly diminished the opportunity for the storm to develop a geostrophic setup due to shore parallel winds prior to the surge component generated near the time of landfall and also to allow the surge to propagate inland. Thus it seems likely that the ADCIRC results are an underestimate of the surge levels that would have been obtained had a larger, slower moving storm been initiated up to 2 days prior to landfall.

<u>REVIEWER RECOMMENDATION 14</u>: ADCIRC should be run for storms having a larger RMW (as recommended previously), a slower forward speed and initiated a longer time before landfall to determine whether this PMH parameter combination generates a greater PMSS at the STP site than those used in the response to this RAI. The study by Resio (2009) has already considered such a storm population and can therefore be the basis of such an analysis assuming the re-calibrated version of ADCIRC is used.

Attachment 6: STP NC Review sotrm Surge – Resio November 22, 2011

INDEPENDENT REVIEW OF SOUTH TEXAS PLANT (STP) UNITS 3 & 4 STORM SURGE ANALYSIS

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November 2011

Contract:
NRC-HQ-11-P-04-01

1. Introduction

The objective of this report is to provide an independent review to assess the technical adequacy of storm surge analyses performed by the applicant and by the NRC review contractor, Pacific Northwest National Laboratory (PNNL) along with the potential for a combined storm surge and main cooling reservoir breach for the South Texas Plant (STP) site, based upon storm surge analyses performed by the applicant and PNNL. This review is based on documents provided by the NRC, peer-reviewed technical literature and relevant guidance documents published by cognizant federal agencies such as the National Oceanic and Atmospheric Administration (NOAA), the U.S. Army Corps of Engineers (USACE) and the Federal Emergency Management Agency (FEMA). The basis for my review will rely on the applicant's adherence to existing standards for licensing and on the adequacy of the surge analyses in light of more recent information on the hurricane characteristics and storm surges.

This review is separated into five main components

1. An overview of the original work and concerns raised in the non-concurrence opinion;
2. A review of the storm parameter selection and application to modeling;
3. A review of the wind and surge modeling elements involved in the estimation of the Probable Maximum Storm Surge (PMSS) at the STP sites;
4. A review of the adequacy of the storm surge analysis conducted by the applicant and PNNL with respect to specific storm surge levels affecting the dam breach analysis; and
5. A summary of the findings and presentation of the final conclusions.

2. Overview of the original work and issues raised in the non-concurrence

The documentation of the original work is extremely sketchy and extracting the information needed for this review from such a terse description has been challenging at times. Some of the applicants' methods are not totally consistent with the older standards nor are they consistent with those used in several recent well-documented studies conducted by federal agencies. A good example of this is the wind field method used in this application. Although we requested and received some documentation providing a very brief justification for their choice, it would have been far better if that had selected one of the methods that has already been applied to a large study, since such models have already been carefully calibrated/validated in conjunction with surge models for the several historical storm surge events. Lacking such validation, one must hypothesize that their method does not introduce a strong bias compared to the previous methods. Although I tend to believe that this would be the case, I had to rely heavily on my knowledge of past modeling results from calibrated/validated modeling systems relative to the results obtained by the applicants and PNNL in order to ascertain the adequacy of their results.

Another general problem that I have with this application is that we now know much more about hurricanes affecting the Gulf coast of the United States than we did in the 1970s when NWS 23 was written. Furthermore, much of what we have learned is not consistent with the older concepts inherent in the NWS 23 approach to storm surges. Although this is not strictly the applicants' problem, this situation needs to be resolved in order to avoid continued issues such as raised in the non-concurrence opinion by Dr. Hosung Ahn. In think that the inclusion of 1970s technology into critical decisions being made today must be taken with a large dose of skepticism, particularly if it is found to be inconsistent with our more recent experience.

I would also like to note that some of the information included in the applicants submission, such as the results of the "SURGE" model combined with the HEC-RAS model (Section 2.4S.5.2.3.1) is so outdated that results obtained from such a methodology should be considered irrelevant for the purposes of a establishing a credible surge level at the STP sites. A reliable estimate can only be made via numerical models which dynamically simulate the coupled processes of storm surge generation and the propagation of the coastal surge inland. A simplistic "bathystrophic" surge model combined with HEC-RAS is not the correct tool for storm surge estimation and is no longer recommended by USACE or FEMA for the type of application in which it used by the applicants. If it is important to link the HEC-RAS model to storm surges, this should be done in a more technically valid fashion.

A final point that should be included in this overview is the need to understand and quantify all uncertainties inherent in the storm specification, the modeling tools, and the site response characteristics when faced with critical decisions. The approach that is followed in the FSAR and SER, along with most of the review to date, has focused on the PMSS within the context of the specification of a deterministic value – with negligible uncertainty in its specification. Since we know that both aleatory (sampling) and epistemic (lack of knowledge) uncertainty are significant even at the 100-year design level (as recognized in the USACE design procedures for the new levee system in New Orleans), there is no reason to expect that such uncertainty will vanish at much, much longer return periods. Since the probabilistic equivalent of the PMSS is hypothesized to fall in the vicinity of an event with an annual exceedance probability of no more than 1 in 1,000,000, such uncertainty obviously not negligible.

3. Review of the PMH parameter selection

For the PMH parameters, the applicants have followed NWS 23 reasonably; however, as noted in the non-concurrence opinion, these must be checked against what we have learned since NWS 23 was published in 1979. There are six parameters used in modern storm surge modeling: 1) storm intensity (a function of peripheral pressure minus the central pressure), 2) storm size (related to the distance from the storm center to the location of maximum winds), 3) storm forward speed, 4) storm track angle relative to the coast, 5) landfall location and 6) the peakedness of the wind speed distribution within the storm (related to a parameter introduced by Holland (1980) – termed the Holland B parameter. The PMH information presented by the applicants in the FSAR contains values or a range of values for three of these parameters in Table 2.4S.5-2 (storm intensity, storm size and forward speed). The SER introduces a set of track directions and sets the landfall location to be co-located with the point of maximum wind speeds at landfall. This leaves only the Holland B parameter which is tacitly assumed to have a value of 1 in their approximations.

In the FSAR, the applicants discuss their application of the SLOSH model for STP sites 3 & 4. The SLOSH model was developed under NOAA funding as a forecast tool and many of the approximations within it are warranted only by the need for fast computer runs during the time period when computers were considerably slowly and had much less storage capacity than available to modern computer systems. The SLOSH model is not believed to be the proper model to use for modern applications in which flooding extends far inland, such as the case being investigated here. It is therefore very good to see that the applicants supplemented their SLOSH runs with ADCIRC runs since this model has undergone considerable calibration and validation in recent years for both coastal and inland sites as part of very large studies conducted by USACE and FEMA.

Besides neglecting the Holland parameter in their analyses, the applicants also neglect variations in storm intensity and size as they approach the coast. Such variations have been found in virtually all modern studies of hurricane characteristics and have been incorporated into all Gulf coast design and insurance studies conducted by USACE and FEMA. As will be shown here, this neglect is somewhat fortuitous, since it adds a degree of conservatism which compensates for some under-conservative parameter choices elsewhere.

The pressure differential in the FSAR/SER analyses (in millibars) is 1020-887=133. The value of 887 mb for central pressure is actually not very much lower than what has been observed in the relatively short period (approximately 60-year interval) where information from airborne instrumentation has been available to provide direct measurements of central pressure. For example, Hurricane Rita was estimated to have had a lowest central pressure inside the Gulf of Mexico of 895 mb and Hurricane Wilma, just before it was impacted by the Yucatan Peninsula when entering the Gulf of Mexico, had an estimated central pressure of only 882 mb. It might be prudent to take the lowest pressure within hurricanes in the Gulf of Mexico might as 880 mb, instead of 887 mb, since the 887 value is clearly not very conservative by modern standards. However, a more typical value for the peripheral pressure in the Gulf of Mexico might be taken as 1015 mb, rather than 1020 mb, so the total impact on the pressure differential of changing these two factors would be about 2 mb which is probably relatively negligible.

A more significant problem emerges when we examine the value of the storm size used by the applicants. As seen in Table 2.4S.5-3 of the SER, the largest storm size simulated is only 20.9 nm. This is only slightly large than the median value for storms along the Texas coast and certainly cannot meet the standard that it represents a conservative value. Irish *et al.* (2008) have shown that storm size is a critical hurricane parameter which strongly influences storm surge levels at the coast. In this context, the selection of a 20.9 nm value for the radius to maximum winds is clearly inadequate for the estimation of the PMSS.

The PMH values used for track angle and forward storm speed appear quite reasonable for the application here, particularly since studies have show these parameters to have only a secondary influence on storm surges (Resio *et al.*, 2009, Niedoroda *et al.*, 2010).

The review to this point has shown that the parameter selection for storm angle, storm landfall location, and forward speed are probably adequate for the simulation of the PMSS. On the other hand, the neglect of the Holland B value and temporal variations in storm characteristics during approach to the coast and the selection of a relatively non-conservative value for storm size all appear to be inconsistent with modern storm surge studies (IPET, 2009) and must be examined further.

Two things are clear concerning the applicants' choice of the storm size. First, it is clear that larger storms have occurred in the hurricane record (for example Hurricane Carla had a radius to maximum winds of about 35 nm during its approach to the Texas coast); and second, it is clear that increasing the size parameter will produce an increase in the storm surge levels at STP 3 & 4. Thus, if there we no compensating source of additional conservatism, I believe it would be impossible to accept the applicants ADCIRC simulations as being adequate for estimating the PMSS.

The neglect of the Holland B parameters would seem to fall into the category of omission which might also create a tendency to be under-conservative in the applicants' submission; however, since the average value of the Holland B parameter in the Gulf of Mexico is only about 1.3, the impact of neglecting this term is likely to be relatively small in this basin. Furthermore, during the IPET study, a specific study was conducted to investigate the impact of the Holland B parameter on storm surges along the coast. Overall, the conclusions suggested that variations in the Holland B parameter tended to play a smaller role than expected on the storm surges generated by a hurricane. Since the radial wind profile shown by the applicants appears to be conservative and since larger storms tend to have lower Holland B values (IPET, 2009), I do not believe that the impact of its neglect will be too serious for this application. This being said, the neglect of the Holland B has not been shown to provide any additional conservatism to compensate for the storm size issue raised previously, in spite of the slightly conservative method of implementing this term into the radial wind distribution.

The remaining inconsistency between the older NWS approach and the modern approach to storm surge modeling is the neglect of temporal variations in storm characteristics in the previous NWS approach. As shown in IPET (2009), as a storm approaches the coast, two primary attributes change – the intensity diminishes and the size increases. These two variations impact surges in opposite directions. A decrease in storm intensity will naturally diminish the surge; while an increase in storm size will naturally tend to increase the surge. The question that must be addressed is to what degree do each of these factors influence surges levels. The answer to this is addressed somewhat in Irish and Resio (2010) where it is shown that storm surges tend to behave asymptotically with storm size, while they do not appear to have an equivalent asymptotic limit with storm intensity. The parametric variation specified by USACE in the IPET study suggests that a storm along the Gulf Coast will tend to increase in size by about 30% over the last 90-nm distance to the coast. On the other hand, a storm with a value of 20.9 nm for its radius to maximum winds at the point where it begins to undergo near-coast evolution will have a central pressure which increases by 14.9 mb before making landfall. Based on numerical simulations of intense storms on a 1:10,000 slope (some of which are shown in Irish *et al.*, 2008), it can be shown that storm surge tends to vary linearly with storm size for storms up to about 25 nm and then asymptotically increases until a limiting value is reach in the range of 30 – 35 nm.

If we take value of 35 nm for a conservative size estimate, the question can now be posed in the following manner: what is the combined effect of neglecting a 14.9 mb increase in central pressure at landfall compared to the utilization of a 20.9 nm value for the radius to maximum winds versus a 35 nm value of this parameter. Simulations included in the work by Irish *et al.* (2008) show that on a slope of 1:10,000 the expected over-prediction (conservatism) in surge due to the neglect of storm decay in the pressure differential specification would be in the range of 8 – 12 %, while the under-prediction (non-conservatism) associated with the selection of a radius to maximum winds that is 14.1 nm smaller than what today would be termed a conservative value would be in the range of 7 – 10 %. This means that, at least from the specification of the parameters for the PMH, the applicants' values obtained via their ADCIRC simulations will likely be consistent with values obtained via a more modern approach.

3. Review of wind and surge modeling elements used for estimating the PMSS at the STP sites

If the applicants had not conducted the ADCIRC simulations, I am not sure that I would have much positive to say in this section. The information obtained via the "SURGE-HEC-RAS" simulations is inappropriate for the estimation of coastal surges and their propagation inland. It uses a simple (one-dimensional) transect with assumed parallel depth contours to represent the bathymetry in an area with obviously non-straight, non-parallel contours. It also assumes that the friction coefficient is constant across its entire domain. Even though the bottom friction factor is "calibrated," it is calibrated in the simplistic context of 1971 technology. Any agreement or disagreement with results obtained by alternative methods is likely due to the subjectivity in the calibration phase of this procedure. Similarly the Halff HEC-RAS storm surge estimates are not acceptable for serious applications in coastal areas, where wind effects, waves, and overland flows all complicate the application of backwater curves.

The storm surge analysis with SLOSH is an improvement over the simplistic nature of the Hydraulic Modeling approach; however, even this application is over-simplified compared to modern storm surge modeling efforts, which is somewhat surprising given the critical nature of this application. For example, it is now well known and accepted in the scientific and engineering community that bottom friction should not be treated as a single constant value across an entire domain as done in the application of SLOSH to STP Sites 3 & 4 in this application. Although the SLOSH model has been shown to be relatively accurate in comparisons conducted by Jarvinen and Lawrence (1985) the accuracy of these comparisons is extremely dependent on the value of the offshore boundary conditions, which is allowed to vary somewhat as a fitting function in typical SLOSH simulations. And, without including the "appropriate" offshore boundary conditions, the SLOSH model usually under-predicts the surges, due to its limited offshore spatial domain. It is not at all clear that the general value for the water level at the offshore boundary from previous SLOSH simulations would be appropriate for very extreme storms, such as the PMH. Furthermore, the lack of inland extent of the SLOSH model and lack of inland resolution are expected, from fundamental considerations, to lead to spurious over- and under-predictions throughout an inland area. For all these reasons, it is difficult to appreciate the applicability of the SLOSH simulations for STP Sites 3 & 4.

The selection of the parameters used in the application of the ADCIRC model to STP Sites 3 & 4 seems to be somewhat inconsistent with currently accepted and reviewed methods being utilized in modern studies; however, as noted previously, the impact of all inconsistencies combined in terms of estimated surge levels should not be too large. The details of the model application seem fairly straightforward, so it seems that the estimates obtained from this approach are likely to be far more valid than the estimates obtained from the first three methods ("SURGE" plus HEC-RAS, "SURGE" plus Halff-HEC-RAC, and SLOSH). The staff's more conservative estimates of the maximum PMH are probably the most reasonable estimates among the entire set of estimates, if one includes the uncertainties involved, and that the estimates obtained by the applicants should be considered consistent with estimates which might be obtained via careful deterministic methods.

4. Review storm surge analysis with respect to specific storm surge levels affecting the dam breach analysis

I concur with the staff's conclusion that the effects of wave set-up and run-up on the main cooling reservoir embankment, based on a deterministic analysis of the PMSS.

5. Summary of findings and final conclusions

I have examined the information presented in the FSAR and SER, along with supplemental files supplied by the NRC. My main finds are summarized below.

1. The overall level of careful documentation included in the FSAR and the SER is very minimal. Some of the tools (such as the "SURGE" HEC-RAS and the "SURGE" Halff-HEC-RAS methods) utilized in this study are inappropriate for coastal surges. To my knowledge they have never been proven to work well in any reviews conducted by coastal surge experts. They give results which are significantly lower than those obtained in applications with modern numerical tools. For these reasons, I did not consider results from this tool to be appropriate for application to the estimation of PMH coastal surge levels in general nor at STP Sites 3 & 4 specifically.
2. The application of the SLOSH model to estimate PMH surge levels is certainly an improvement over the very outdated "SURGE" model; however, the lack of careful quantitative methods to estimate water levels at the offshore and lateral boundaries of the SLOSH grid make the application of this model to the PMH situation very questionable. Also, the lack of inland extent and lack of resolution of typical SLOSH grids make them a poor choice to utilize for determining surge levels in a PMH flooding event.
3. The ADCIRC runs should have been conducted with wind fields for which the ADCIRC model has been calibrated in recent large studies. The use of a newly postulated wind model, although perhaps conservative in its design, was an unnecessary complication to my effort to quantify the sum of all of the different effects on the final estimated surge levels. However, since the effect of the new method was to force all the solutions to have the same maximum wind speed, I believe that the overall effect of using this new tool, compared to those for which extensive calibrations are available, should be fairly small.
4. The use of a radius to maximum wind speed of only 20.9 nm is much too small by modern standards. Although this value may be consistent with Figure 2.5 in NWS 23, it is not consistent with observations from the two most powerful storms striking the Texas coast in the last 60 years (Hurricanes Carla and Ike), which both had radii of maximum winds substantially larger than the selected value, nor is it conservative with respect to the overall distribution of storm sizes along the Texas coast. Inclusion of larger value for the radius to maximum wind speed in the PMH would produce a net increase in the estimated storm surge levels in the PMH.
5. The applicants did not include the effects of storm intensity decay as they approach the coast. Inclusion of this term would produce a net decrease in the estimated storm surge levels in the PMH.
6. Based on publications (Irish *et al.*, 2008; Resio *et al.*, 2009; Irish and Resio, 2010) and simulations which contributed to those publications, the net effect of all of the deviations in the treatments of storm size and pressure differential is expected to be fairly small – only because the two non-negligible terms of opposite sign ended up having about the same magnitudes. Clearly, this is somewhat fortuitous and cannot be depended on for future licensing decisions.

In conclusion, I concur with the staff's decision on the licensing at this site; but only given the perspective of a deterministic framework for PMH/PMSS estimation. Since we have an extremely small sample duration relative to the mean return period for which our estimates are made and since there are many unknowns in our modeling of storm surges, it only seems prudent to include some level of uncertainty into these design levels. In any event, it is hoped that this review will encourage the NRC to continue their ongoing effort to establish a newer set of guidelines for licensing considerations in coastal areas, one that stresses the application of tools that have shown to be calibrated and function well within a particular area – rather than allow the application of outdated methods and outdated data to persist.

6. References

Irish, J.L., Resio, D.T. and M.A. Cialone, 2009: A surge response function approach to coastal hazards assessment. Part 2: Quantification of spatial attributes of response functions, Nat. Hazards, 51, 183-205.

Irish, J.L. and D.T. Resio, 2010: A hydrodynamics-based surge scale for hurricanes, Ocean Engr., 37, No. 1, 69-81.

Niedoroda, A.W., Resio, D.T., Toro, G.R., Divoky, D., Das, H.S. and C.W. Reed, 2010: Analysis of the coastal Mississippi storm surge hazard, Ocean Engr., 37, No. 1, 82-90.

Resio, D. T., Irish, J.L., and M.A. Cialone, 2009: A surge response function approach to coastal hazards assessment – part 1: Basic concepts, Nat. Hazards, 51, 163-182.

IPET (Interagency Performance Evaluation Task Force), 2009: Volume 8, USACE.

Attachment 7: STP NC Review MCR Breach – Wahl
November 22, 2011

RECLAMATION

Managing Water in the West

Review of Main Cooling Reservoir Embankment Breach Analysis

Expansion of South Texas Project Electric Generating Station

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U.S. Department of the Interior
Bureau of Reclamation
Technical Service Center
Hydraulic Investigations and Laboratory Services Group
Denver, Colorado

November 2011

Mission Statements

The mission of the Department of the Interior is to protect and provide access to our Nation's natural and cultural heritage and honor our trust responsibilities to Indian Tribes and our commitments to island communities.

The mission of the Bureau of Reclamation is to manage, develop, and protect water and related resources in an environmentally and economically sound manner in the interest of the American public.

Disclaimer

The information provided in this report is believed to be appropriate and accurate for the specific purposes described herein, but users bear all responsibility for exercising sound engineering judgment in its application, especially to situations different from those studied.

This review was requested and funded through an interagency agreement with the U.S. Nuclear Regulatory Commission, Office of Nuclear Regulatory Research.

CONTENTS

TABLES

FIGURES

Introduction and Purpose

A proposal to add two nuclear reactors at the existing South Texas Project Electrical Generating Station has prompted the need to analyze potential breach of the Main Cooling Reservoir embankment at the plant in order to establish the design basis flood (DBF) level for safety-related facilities. Analysis of the potential breach is contained in several documents, including but not limited to:

- Final Safety Analysis Report (FSAR) prepared by the applicant;
- Safety Evaluation Review (SER) prepared by staff of the U.S. Nuclear Regulatory Commission (NRC) and the Pacific Northwest National Laboratory (PNNL). PNNL serves as a contractor to NRC.

A non-concurrence letter dated June 8, 2011 (Ahn 2011a) raises questions about the accuracy and conservatism of the embankment breach analyses described in the SER, Chapter 2.4. Additional material in the non-concurrence package includes:

- Enclosure 1: Re-Analysis of the MCR Breach Flood (Ahn 2011b);
- Marked up package of calculation worksheets first provided by PNNL, containing corrected model input files, analyses, and results supporting the points of the non-concurrence letter (Ahn 2011c).

Subsequently, PNNL replied to the non-concurrence letter with a document titled *Response To: Re-Analysis of MCR Dam Breach Flood (June 20, 2011)*, and also provided an electronically marked-up PDF copy of the *Re-Analysis of the MCR Breach Flood* report.

The NRC Office of Nuclear Regulatory Research requested this independent review of the reports listed above and other supplemental materials provided by NRC for the purpose of resolving the questions raised by the non-concurrence letter.

Scope

The non-concurrence letter raised issues in three major categories:

1. Main Cooling Reservoir (MCR) breach analysis,
2. Flood analysis of hurricane and MCR breach combination, and
3. Maximum groundwater level.

Specific issues related to the MCR embankment breach are addressed in this review. This review also addresses the question of whether MCR embankment breach combined with hurricane storm surge is a credible flooding scenario. For this purpose, this reviewer accepts the validity of provided hurricane storm surge analyses. Other reviewers are addressing specific questions related to hurricane storm surge modeling. Maximum groundwater level issues are not addressed in this review. A detailed list of the issues considered in this review is given in Table 1.

Table 1. — Summary of non-concurrence issues.

ISSUES
Incorrect choice of empirical breach parameter regression equations. • Use of equations outside of valid ranges, • Exclusion of more conservative equations, and • Lack of recognition of similarity to levees that have historically experienced very wide breaches.
Incorrect application of NWS-BREACH model for confirmatory analysis. • Claim that NWS-BREACH model is discredited by many hydrologists • Incorrect selection of Manning's *n* roughness value, • Embankment width (crest length) and tailwater channel were restricted in NWS-BREACH model input, which constrained prediction of breach width and peak outflow, and • Lack of consideration for effects of scour hole, concrete blocks, and sand core.
Inappropriate comparison to historical case of the breach of Martin Cooling Pond
Is combined hurricane storm surge and MCR embankment breach a credible failure mode?

MCR Embankment Description

The Main Cooling Reservoir (MCR) embankment for the South Texas Project forms a complete ring containing the cooling reservoir serving the existing project. The embankment is approximately 45 ft high, 12.4 miles long and encloses an area of about 7,000 acres (3 miles by 4 miles). Water circulates within the MCR in a counterclockwise direction during normal operations, controlled by interior dikes having a total length of approximately 36,300 ft. Crest elevation of the perimeter embankment varies between El. 65.75 ft and 67 ft.

Normal maximum MCR operating level is El. 49 ft above mean sea level (MSL), and minimum operating level is El. 25.5 ft MSL. The probable maximum flood level within the MCR is El. 52.1 ft MSL. The postulated failure mode affecting the design basis flood level for plant expansion is internal erosion due to seepage through the embankment caused by a failure of its internal drainage system. This failure would occur at a reservoir level of 50.9 ft MSL (storage approximately 195,000 ac-ft). The starting water level in the MCR considered for the breach analysis was 50.9 feet. This level is described in the FSAR as the level corresponding to the response of the MCR to one-half PMP on the normal maximum operating level plus the effect of wind set-up produced by the 2-year wind speed (50 mph) from the south.

Geotechnical sections for the embankment are shown in Figures 1 and 2. The interior face of the embankment is protected by a 2.5-ft thick soil-cement layer to protect against wave erosion. Other materials used in embankment construction are described in the STPEGS UFSAR document, section 2.5.6.4.1.3:

> *"Zone A fill is clay with a minimum LL of 40 and was the material utilized to fill the inspection trench, to fill miscellaneous areas where former stream courses cross the embankments or dikes, and as fill at the spillway structure. Zone B fill is clay with a minimum LL of 30 and is the material utilized as fill at all locations in the embankment other than the inspection trench, the interior and exterior berms, the sand core, and the sand drainage blanket. Zone C fill is nonplastic granular soil with not more than 35 percent passing a no. 200 sieve and is the material utilized for the sand core and the sand drainage blanket. Zone D fill is zone A, B, C or any combination thereof and is the material utilized for the interior dikes and the interior and exterior berms of the embankment. Type I fill is sand conforming to ASTM C 33-74 fine aggregate and is utilized for filters around collector pipes and structures. Type II fill is gravel conforming to ASTM C 33-74 coarse aggregate and is utilized for filters around collector pipes and structures."*

Discussion of Non-Concurrence Issues

Initial Dam Breach Modeling Approach

To address the inherent uncertainty in the modeling of an embankment dam breach, a variety of modeling approaches were applied in the FSAR and SER. These are reviewed here to place the subsequent discussion in context.

The applicant reported in the FSAR (section 2.4S.4.2.2) that breach parameter regression equations that statistically relate breach parameters to general embankment and reservoir properties were used to make estimates of the average breach width and the time required for breach development. The specific equations utilized were developed by Froehlich (1995b) for breach width, and MacDonald & Langridge-Monopolis (MLM) (1984) for breach time. These breach parameters were then supplied as input parameters to the National Weather Service FLDWAV model (Fread 1993) which was used to predict the resulting breach outflow hydrograph. This yielded a peak discharge of 130,000 ft^3/s. The breach outflow hydrograph was then provided as input to the RMA2 model to predict flood inundation depths and flow velocities at locations of interest. In addition, a verification check was made using a regression-based equation to predict peak breach outflow directly (Froehlich 1995a). This equation predicted a peak outflow of 62,600 ft^3/s. [Note that Ahn (2011b, pg. 7) misattributes this result to the National Weather Service BREACH model, and the SER misattributes the Froehlich peak flow equation to Wahl (1998). The equation was developed by Froehlich (1995a) and was only discussed by Wahl (1998).]

The applicant also made a confirmatory analysis using the National Weather Service (NWS) BREACH model (Fread 1991), a physically-based model that simulates erosion processes for embankment dams and predicts the breach outflow hydrograph. The sensitivity of the model to various input parameters was tested, and the Manning's *n* roughness value for the breach channel was found to be the most important parameter. Values ranging from n=0.025 to n=0.080 were tested, and a value of Manning's n=0.050 was used for a final run that predicted a peak breach outflow of 83,200 ft^3/s.

The SER confirmed the calculations of breach parameters and peak outflow from regression equations, but did not repeat the FLDWAV model run. PNNL also made a confirmatory analysis using NWS-BREACH, and performed additional sensitivity testing. Again, the Manning's *n* value for the breach channel was found to be the most important parameter. A value of Manning's n=0.075 was selected as being conservative and produced a peak outflow of 128,000 ft^3/s, which was used in subsequent 2D modeling of downstream flooding conditions. This was comparable to the 130,000 ft^3/s result from the FLDWAV model reported by the applicant in the FSAR.

Breach Parameter Regression Equations

The FSAR and SER utilized two breach parameter equations, Froehlich (1995b) to predict breach width and MLM (1984) to predict the breach formation time. Wahl (1998) of the Bureau of Reclamation reviewed these equations and numerous others for application to embankment dams impounding storage reservoirs. Wahl (1998) does not make recommendations for or against specific breach parameter equations. Wahl (2004) analyzed the uncertainty of various breach parameter prediction equations.

The Froehlich (1995b) equation yielded an estimated average breach width (e.g., top and bottom of the breach) of 417 ft. The MLM (1984) equations estimate both the volume of embankment material removed and the time required for failure. The result for the time of failure was 1.7 hours using the MLM equation, which is an envelope equation that matches the lower bound (shortest breach times) of a plot of breach times versus predicted volume of material eroded. The breach time data on this plot are described by MacDonald and Langridge-Monopolis as estimates of the "maximum times that it could have taken for the breaches to develop". The MLM (1984) equation was chosen because it gave a conservatively fast breach in comparison to other equations that were considered.

Ahn (2011a; 2011b) makes several assertions related to the breach parameter predictions made using regression equations:

- The Froehlich (1995b) equation is inappropriate for this application because the predicted breach width is greater than 164 ft.
- The equation for eroded material volume given by MLM (1984) can be used to estimate breach width and would yield a much greater breach width.
- Larger breach widths are plausible based on historic experience, including several cases of levee breach.
- The use of the Froehlich (1995b) equation for breach width and the MLM (1984) equation for breach formation time is inappropriate because it is a mix-and-match approach.

Use of equations outside of valid ranges

Ahn (2011b) maintains that the Froehlich (1995b) equation should not be used in situations where it predicts a breach width greater than 164 ft. The statement is made that:

> *"USBR (1998) states that the Froehlich's [sic]equation appears to be the best predictor for the breach widths less than 164 ft because the breach width records used to develop the Froehlich equation are small. In other words, Froehlich's breach width equation is not applicable to the MCR breach case."*

The USBR (1998) document referred to above is the Dam Safety Office research report DSO-98-004 by Wahl (1998). Wahl (1998) compares the ability of three breach width equations to match observed values from case studies of dams of all sizes exhibiting a range of breach widths. For cases of observed breach width less than 50 m (164 ft), the Froehlich (1995b) equation appears to match observed breach widths better than the other two equations. For cases of observed breach width greater than 164 ft, all three equations perform similarly and Wahl (1998) makes no comment on the best choice of equation for this range. Wahl (1998) never states that the Froehlich equation is inappropriate for the larger range of breach widths, only that it appears to be the better choice in cases where the observed breach width is small. The inference is incorrect that Froehlich (1995b) is inappropriate for breach widths greater than 164 ft just because it is the better choice for breach widths less than 164 ft.

Ahn (2011b) also reports that Wahl (1998) observes that "the breach width records used to develop the Froehlich equation are small." Wahl (1998) analyzes the characteristics of a collection of 108 dam failure case histories compiled from numerous studies of dam breach, including that of Froehlich (1995b). This analysis concludes that small dams (with commensurate small breach widths) do comprise the majority of the database, but this analysis is only performed on the entire database, not specifically on the cases cited by Froehlich (1995b). The fact that large dams are underrepresented is a fault common to almost all compilations of dam failure case history data, including MLM (1984) and Froehlich (1995b).

Exclusion of more conservative equations

Ahn (2011) asserts that the MLM (1984) equation should have been used to estimate the MCR breach width and would have produced a larger, more conservative result. The equation referred to computes the best estimate of the volume of eroded embankment material, not the breach width itself. However, this equation can be used to estimate the breach width. The MLM (1984) equation was not included in the comparison of breach width equations by Wahl (1998), but was included in the comparison by Wahl (2004). In that latter comparison, the uncertainty of breach width predictions using the MLM (1984) equation (±0.8 orders of magnitude) was twice that of predictions made using Froehlich (1995b) (±0.4 orders of magnitude). This justifies the selection of the Froehlich (1995b) equation.

Levees vs. dams

In arguing that larger breach widths are plausible for the MCR embankment, Ahn (2011b) cites numerous case histories of large breach widths, including several levees. Ahn's discussion of the issues also uses the term *levee* repeatedly. PNNL (2011) disputes the characterization of the MCR embankment as a levee, stating that levees are more prone to large breach widths for numerous reasons that do not apply to the MCR embankment, including:

1. Levees generally are used to contain discharges within stream channels and fail by overtopping when flood levels exceed the design bases. When large floods cause levee failures, the flood overtops the levee along significantly long distances over the length of the levee.

2. The duration of overtopping also can be significantly long, depending on the magnitude of the flood compared to the levee's design basis.

3. A breached levee also experiences substantial shear forces from the flood discharge as the river flows along its length and head cutting from floodwaters that overtop the levee. These conditions promote a longer levee breach.

It is the reviewer's opinion that levees do typically experience larger breach widths, primarily for the second reason cited above. The boundary condition at the upstream side of a levee is typically a river, ocean, or other live water body capable of maintaining a large driving head as the breach develops, with the quantity of water available far exceeding the amount that flows through the breach. This condition is irrespective of whether the breach is caused by overtopping or internal erosion. In both cases, breach will initiate at a specific point along the embankment, either at the site of the internal erosion or at the site where overtopping first causes a headcut to advance through the embankment. Even though initial overtopping may occur over an extended length of the crest, the headcut that breaks through the embankment will be at one specific site, either a low spot in the crest where the unit overtopping discharge is high, or at a weak spot associated with materials variability. It would be rare for a second headcut to break through the embankment at another location, since the upstream reservoir will begin to drop as soon as flow through the first breach opening begins to increase significantly. Once initiated, the breach width will then increase as long as the upstream head can be maintained at a level that sustains hydraulic attack on the sides of the breach opening exceeding the threshold for breach widening. In many levee situations, this time can be very long, because an effectively infinite body of water or live flow sustaining a high river level is available.

In contrast, a dam typically impounds a reservoir of finite volume, so the upstream water level can diminish significantly as the breach opens and the upstream reservoir drains. Thus, the availability of water limits the potential for a large breach width, even if the site geometry (presence of dam abutments) does not. In cases of hydrologic-driven dam failures, the upstream reservoir can be maintained for a longer period and would approach the typical levee scenario, but in general the dam failure scenario involves a limited volume of water and there is a likelihood that the upstream head will diminish as the breach width increases. In this respect, the MCR embankment is much more like a dam than a levee, since the volume of water contained in the impoundment is finite and there is no potential for inflow to the impoundment, other than direct precipitation over the surface area of the reservoir. The design and construction characteristics of the MCR embankment are also more dam-like than levee-like (engineered foundation and embankment sections, filter zones, careful construction, etc.) From a materials standpoint, the MCR embankment is also much more like a typical dam than a typical levee, since it is composed primarily of plastic clay soils, unlike many levees that were constructed from available on-site materials, regardless of suitability for retaining water or resisting erosion.

Use of a mix-and-match approach

Ahn (2011) objects to the mix-and-match approach of using the Froehlich (1995b) relation to predict breach width and the MLM (1984) equation to predict breach formation time. Alternatives might have been to utilize either the MLM (1984) equations for both parameters, or the Froehlich (1995b) equations for both parameters. However, having established that the Froehlich (1995b) breach width equation is superior to the MLM (1984) breach width equation by virtue of its significantly lower uncertainty (Wahl 2004), the exclusive use of the MLM (1984) equations would have been a poor choice. The use of the Froehlich (1995b) breach time equation would have produced a failure time estimate of nearly 8 hrs according to Ahn's Table 6.3-1. In contrast, the MLM (1984) breach time equation yields an estimate of 1.7 hrs, which is far more conservative. Ahn (2011) also suggests the use of a best-fit breach time equation based on reanalysis of data from MLM (1984), but this equation yields a slower breach (2.54 hr) than the original MLM (1984) equation. It should be emphasized that the breach time equation given by MLM (1984) is a *lower* envelope equation (fastest breach time) fitted to the bottom edge of data described as maximum times that could have been required for breach to occur. The only equation shown in Ahn's Table 6.3-1 to have a shorter breach time is the Bureau of Reclamation (1988) equation, which estimates a breach time of 0.66 hr. This rapid breach time is the result of the associated prediction of a breach width of only 66 ft. In summary, the selection of use of the mixed breach parameter equations effectively utilized those equations with demonstrably smaller uncertainty while maintaining appropriate conservatism.

NWS-BREACH modeling

Is the NWS-BREACH model discredited?

Ahn (2011a, 2011b) states:

> *"The BREACH model is based on the Meyer-Peter & Mu[e]ller (MPM) sedimentation equation. However, many hydrologists have noted that the MPM method is not adequate for dam breach conditions as it is developed for estimating sedimentation in small mountain areas. The MCR breach condition is quite different from the mountain condition in terms of slope and bottom condition."*
>
> *"Because the result of BREACH model is highly depending [sic] on the model inputs, many hydrologists discredited the use of the BREACH model (e.g., Wahl, 1999). Also, the author (Dr. Fread) of the BREACH model recommends that BREACH model be used [for] supplemental purpose only."*

The actual basis for erosion modeling in NWS-BREACH is the Meyer-Peter and Mueller equation as modified by Smart (1984), applied to flow in an idealized channel paralleling the downstream dam slope. Although intended for application to mountain streams, the equation was developed from laboratory flume tests in channels having slopes up to 20%. The flume environment is not dissimilar to the conditions that might exist in a developing breach channel. Although breach channel slopes could exceed 20%, this equation was state-of-the-art at the time of its incorporation into the NWS-BREACH model.

The citation for Wahl (1999) is missing from Ahn's reference list. In fact, such an article by this author in the year 1999 does not exist. Other potential citations for the discrediting of the NWS-BREACH model might be Wahl (1997) or Wahl (1998). However, none of these articles "discredits" the BREACH model. They do point out the shortcomings of the NWS-BREACH model in discussions of improvements that could be made to produce a next-generation embankment erosion and breach modeling tool. They also note that NWS-BREACH was the current state-of-the-art at the time and the most widely used and accepted physically-based breach model. Efforts to produce next-generation models that utilize different erosion algorithms and better conceptualization of breach mechanics are presently underway. Research versions of new models (Temple et al. 2005; Hanson et al. 2005b; Mohamed 2002) do exist, but none have yet become widely accepted for practical use. The WinDAM B model (Hanson et al. 2011) developed by the USDA-Agricultural Research Service is being released at the time of this writing, and it would be a recommended modeling tool in the future, although at present it addresses only breach due to overtopping flow (internal erosion simulation is still under development).

The greatest indication of any discrediting of the NWS-BREACH model is the fact that it can no longer be downloaded freely from the National Weather Service web site and is no longer available commercially from the BOSS Corp., which has recently gone out of business. The National Weather Service also no longer distributes the FLDWAV model used to route dam-break floods to determine downstream inundation. The stated agency policy is to concentrate future development on the use of the U.S. Army Corps of Engineers HEC-RAS modeling system. HEC-RAS is functionally similar to FLDWAV, with the ability to simulate breach outflow through a breach opening that is prescribed to develop at a specified rate to a specified size. The size and rate are typically provided from the use of breach parameter regression equations. The HEC-RAS model does not contain a module to simulate embankment erosion and breach development in a physically-based manner, so it does not replace the functionality of the NWS-BREACH model. Thus, for now, NWS-BREACH remains an established state-of-the-practice tool, albeit one that is now difficult to obtain.

The recommendation by Dr. Fread that NWS-BREACH be used only for supplemental purposes seems consistent with the manner in which the model has been applied by the applicant and PNNL to this project. The primary basis for breach outflow determination is the application of breach parameter equations and FLDWAV. NWS-BREACH is used to make the confirmatory analysis.

Selection of Manning's n

Manning's *n* values in NWS-BREACH

The FSAR and SER both note the sensitivity of NWS-BREACH results to the selection of Manning's *n* values. Tejral et al. (2009) also confirms this finding. Two different values of Manning's *n* must be provided to the NWS-BREACH model. One is integrated into the equations that describe flow through the breach and the calculation of erosion rates via the Meyer-Peter and Mueller equation. The other value of Manning's *n* is used to compute flow conditions at the tailwater section, downstream from the breach, which can cause a backwater effect that retards flow through the breach opening. It is the first of these two *n* values that has a profound effect on the predicted breach outflow hydrograph through its effect on the simulation of flow through the breach opening and the resulting sediment transport.

NWS-BREACH use of the Strickler equation

The Manning's *n* value for the breach section can be specified in one of two ways in NWS-BREACH (Fread, 1991). Manning's *n* can be specified directly, or the median particle diameter of the embankment material can be provided (D_{50}), and the Manning's n value will be computed internally using the Strickler equation. Manning's n values were directly specified in all NWS-BREACH model runs made for this project, but reference was made by all parties to the Strickler equation when determining appropriate n values to specify (although Ahn 2011b refers to it as Straler's method).

Many versions of the Strickler equation exist in documentation for the NWS-BREACH model and in the general literature. The PNNL calculation package (PNNL 2011b) highlights some of the different equations. The basic form of the equation in all references is:

$$n = a(D_{50})^b$$

where n is the Manning's roughness value, a is a coefficient that varies depending on the application and the units of D_{50}, and b is an exponent. With D_{50} given in feet, values of a are typically around 0.034, with minor variation attributed to the focus of different investigators on different sediment sizes and channels. Strickler's original work (see Chow 1959, p. 206 for an English-language synopsis) recognized that a was somewhat dependent on the relative roughness of the channel, but suggested an average value of 0.0342. With D_{50} given in meters the value of a is typically around 0.047, and when D_{50} is given in millimeters the value of a is about 0.013 to 0.015. Erroneous conversions of the coefficient to other units and variation of its value for different applications and relative roughness situations have created great confusion about the value of the coefficient in this equation.

However, by far the greatest confusion associated with this equation is related to the exponent b. Conflicting values of 0.167 and 0.67 exist in the NWS-BREACH documentation and in recent and older literature. Table 1 summarizes values of the exponent contained in several versions of the equation uncovered by this reviewer.

Reference	Exponent b	Primary source
NWS-BREACH documentation	0.167	Printed copy obtained by this reviewer ca. 1995 from Dr. Danny Fread
NWS-BREACH documentation	0.67	http://rivermechanics.net, downloaded October 2011
Chow (1959)	0.167	Strickler (1923) in German
Yen (1991)	0.167	Strickler (1923) from 1981 translation
Tejral et al. (2009)	0.67	BOSS Corporation NWS-BREACH documentation
NWS-BREACH FORTRAN source code	0.167	Electronic copy obtained by this reviewer ca. 1995 from Dr. Danny Fread

Given the agreement of the older references that trace back to the original work of Strickler, there is no doubt that the correct value of the exponent is 0.167, and this correct value was coded into NWS-BREACH by Dr. Danny Fread. The erroneous 0.67 value appears to have come about as a typographical error when the documentation was converted to electronic (PDF) format and some of the equations were retyped (some equations in the online document appear to use different fonts than the majority of the document). Two sources of PDF files containing the erroneous 0.67 value are (1) the commercial version of NWS-BREACH that was distributed by BOSS Corp. until they recently closed their business, and (2) the http://rivermechanics.net web site. Personal communication with Ronald Tejral of the USDA-ARS verified that he obtained his copy of the NWS-BREACH model and documentation in recent years through BOSS Corp. He verified that the BOSS version of the model in his possession uses the 0.167 coefficient in the FORTRAN source code. A printed copy of the 1988 documentation of BOSS BREACH has the correct 0.167 exponent in the Strickler equation.

The http://rivermechanics.net web site is operated independently from the National Weather Service by a former NWS employee and colleague of the late Dr. Fread, Ms. Janice Sylvestre (nee Lewis). When I contacted Ms. Sylvestre about the discrepancy in the value of this coefficient, she confirmed that the documentation on the http://rivermechanics.net web site was incorrect, but that the model as available from the same web site did indeed use the value of 0.167 in all of its calculations.

Applying the Strickler equation

The correct version of the Strickler equation as given in original NWS-BREACH documentation is

$$n = 0.013\ (D_{50})^{0.167}$$

with D_{50} in millimeters.

Using this equation, values of n corresponding to different particle sizes are as follows:

D_{50}, ft	D_{50}, inches	D_{50}, mm	*n*	*Notes*
3.28E-06	0.0000	0.001	0.004	
2.46E-04	0.0030	0.075	0.008	Fine sand
0.02	0.19	4.75	0.017	Coarse sand/fine gravel
0.16	1.97	50	0.025	Coarse gravel
3.28	39.4	1000	0.041	Boulder size range
10.83	130	3300	0.050	
118.11	1417	36000	0.075	

PNNL's analysis for the SER and the applicant's analysis for the FSAR initially computed values of n based on the embankment's composition of mostly clay materials (D_{50} approx. 0.001 mm), but the values seemed small to the analysts, so amid the uncertainty of the various forms of the Strickler equation, values were increased as high as n=0.080 to evaluate sensitivity over a range that was considered reasonable. A value of n=0.050 was used to obtain a peak breach outflow of 83,200 ft^3/s reported in the FSAR, and a value of n=0.075 was used by PNNL for their final NWS-BREACH model runs which produced a peak outflow of 128,000 ft^3/s reported in the SER.

Dr. Fread's NWS-BREACH documentation states that "the breach flow is adequately described by quasi-steady uniform flow as determined by applying the Manning open channel flow equation...", and the *n* value should represent the channel roughness "based on the average grain size of the material forming the breach channel." While the MCR embankment is composed predominantly of a clay zone, it is likely that materials might detach as aggregates of finer particles, so that the effective grain size of the material forming the breach channel could be somewhat larger than true clay-sized particles. The question is how much increase is appropriate.

Fread (1988; revised 1991) offered four example applications of the NWS-BREACH model. For Teton Dam, the embankment was predominantly fine-grained (CL-ML), and Dr. Fread judged the Strickler equation to not be applicable. Instead, he made a separate analysis using the Moody diagram to obtain what he considered to be a reasonable Darcy friction factor for the material and then selected n=0.013 to produce model behavior consistent with this friction factor. For Lawn Lake Dam he used the Strickler equation with D_{50}=0.25 mm, which yields a value of n=0.010. For the Mantaro landslide dam he used D_{50}=11 mm (n=0.019), and for the Spirit Lake dam formed from volcanic avalanche debris he used D_{50}=7 mm (n=0.018). Dr. Fread obtained reasonable agreement between modeled and observed breach outflows in each case. Clearly, Dr. Fread found Manning's *n* values in the range of 0.025 and smaller to be reasonable. To obtain Manning's *n* values of 0.050 or 0.075 would infer D_{50} values of 11 to 118 ft, comparable to ¼ to 3 times the height of the MCR embankment. The value of 11 ft is perhaps plausible, given the fact that the upstream face of the dam is provided with a 6-ft thick soil-cement protective layer, and blocks from this layer could affect the breach channel roughness, although this layer comprises a small fraction of the total embankment volume. The D_{50} value of 118 ft is not a plausible value. Based on the examples of Fread and calculations made using the correct form of the Strickler equation, a value of n=0.025 appears to be reasonable and conservative. This value yielded a peak outflow of about 31,000 ft^3/s in the original runs made by PNNL. A value of n=0.050 seems extremely conservative, and a value of n=0.075 does not seem credible to this reviewer.

Effects of soil-cement blocks and sand core

Ahn (2011) criticizes the application of the NWS-BREACH model in the FSAR and SER reports for its lack of consideration for the effects of soil-cement blocks and the sand chimney filter zone (described as "sand core") that might affect the breach channel roughness. Ahn proposes estimating the Manning's *n* value for the breach channel by applying a method described in Chow (1959) that utilizes a base *n* value related to the channel materials and additive and correction factors accounting for surface irregularities, channel shape and size variations, obstructions, vegetation, and meandering. The sand chimney is incorporated by Ahn in the base *n* value (although the sand may be so erodible that it is quickly removed and may play no role), while the soil-cement lining of the upstream face of the dam provides blocks that would act as obstructions. Ahn also utilizes other additive factors for effects of channel irregularity, variations in cross section, and vegetation.

The Chow (1959) method is intended for application to uniform flow, and while not explicitly stated, the method is demonstrated, illustrated, and discussed almost entirely in the context of subcritical flow. The method would have been well known to Dr. Fread as a result of his development of the DAMBRK (Fread 1977, 1984) and FLDWAV models (Fread 1993; Fread and Lewis 1998), which route dam break floods through channels that are heavily affected by vegetation, channel meandering, obstructed flow, expansion and contraction, etc. Dr. Fread specifically chose not to use this method or discuss the possibility for its use in the NWS-BREACH documentation because flow through a dam breach opening is not affected appreciably by these additive factors. In contrast to uniform, subcritical flow in which energy loss from vegetation, obstruction, channel irregularity, meandering, etc. are significant in comparison to the inertia of the flow, rapidly varied and highly energetic dam breach outflows have such power associated with them that they scour all such features and their effects out of the breach channel. Vegetation, meanders, abrupt expansions and contractions, and even large and heavy obstructions are forcibly removed, straightened, scoured, and eliminated from the breach outflow channel, leaving only the base *n* value. It is this base *n* value, related to the materials forming the channel, that Dr. Fread intended to model through the use of the Strickler equation. The Chow (1959) method is not an appropriate means for estimating the Manning's *n* value for flow through the breach opening in NWS-BREACH.

Sensitivity of NWS-BREACH to D_{50} and Manning's *n*

All investigators on this project noted that increased values of Manning's *n* produced faster breaches, wider breach widths, and larger peak breach outflows. The mechanism by which this occurs is poorly understood. The basic sediment transport equation in NWS-BREACH is (Eq. 40, Fread 1988, revised 1991):

$$Q_s = 3.64\left(\frac{D_{90}}{D_{30}}\right)^{0.2} P\frac{D^{2/3}}{n}S^{1.1}(D \times S - \Omega)$$

where Q_s is the sediment transport rate, D_{90} and D_{30} are 90th and 30th percentile sediment sizes, P is the wetted perimeter of the flow through the breach channel, D is the hydraulic depth, n is the Manning's n roughness value, S is the channel slope (equal to the downstream embankment slope), the product $D \times S$ is the applied shear stress and Ω is a critical shear stress parameter. Tejral et al. (2009)

speculated that the primary influence of D_{50} and Manning's n was through the n value that appears in the denominator. This would suggest that as D_{50} and n increased, sediment transport would decrease, which should yield a slower, smaller, less severe breach. Tejral et al. (2009) observed this expected behavior from the model for a few small dams (laboratory model-scale), but opposite behavior for most large dams.

The sensitivity to D_{50} and n can be understood when we see that D_{50} and n also exert influence in other terms of the equation, since the value of n is also used in Manning's equation to compute the flow depth, D, and wetted perimeter, P, when there is open channel flow through the breach opening. A sensitivity analysis by this reviewer found that the term $PD^{2/3}/n$ remains nearly constant as D_{50} is varied, but the applied shear stress, $D \times S$, increases significantly as D_{50} and n are increased, since larger values of n produce larger flow depths, D. Larger values of D_{50} also increase the critical shear stress term, Ω, but at a much slower rate than the increase of applied stress, until sediment sizes become extremely large. The net effect is that an increase of D_{50} (and n via the Strickler equation) leads to an increased excess stress (the stress above the Ω threshold) and increased sediment transport rate for most practical values of D_{50}. This is counterintuitive for many because we have an inherent feel for the increase in critical stress that comes from increasing sediment size, but we do not have a similar intuition for the increase in depth and increase in applied shear stress that accompanies a rougher flow boundary.

This behavior occurs when there is open channel flow through the breach, which applies to an overtopping failure and also to a piping failure once it progresses to the point of collapse of the upper part of the embankment. During the initial stages of a piping failure, the depth and wetted perimeter terms are not a function of n, but are determined by the pipe size, so changing the roughness does not have the same affect on applied stress. In fact, increased roughness will reduce the flow through the pipe for a given upstream head, so higher roughness will reduce the applied stress. Increased D_{50} and n will also increase the critical shear stress needed to initiate erosion, so the erosion rate will decrease in response to a larger D_{50}. If the applied stress is significantly greater than the critical shear stress, these effects may be minor and the erosion rate during piping initiation will be relatively insensitive to D_{50} and n. When the piping situation progresses to become an open channel flow through the breach opening, the sensitivity to D_{50} and n changes back to that described above.

Effects of scour hole

Ahn (2011) discusses scour hole formation downstream from levee breaches and reports results of testing the effects of postulated scour holes on NWS-BREACH model runs. The mechanism by which a scour hole increases the peak outflow is not clearly explained. It is speculated that "the scour hole could increases [sic] the capacity (area) of breach outflows..."

Ahn's method for incorporating scour holes into the NWS-BREACH model runs is not fully explained. One may speculate that the tailwater section downstream from the embankment was lowered to represent the geometry of the scour hole, and that this increases the net head through the breach or reduces the chance for backwater to retard the flow through the breach opening. This ignores the fact that the tailwater channel further downstream from the scour hole will produce backwater itself that could submerge the scour hole and negate its effects. In the absence of a clear explanation for how the scour hole was incorporated into the NWS-BREACH model and a proven physical mechanism by which an increase in breach outflow occurs, the results of the scour hole sensitivity runs must be discounted. It is also significant to note that all of these runs were made with the very large Manning's n=0.075. It was concluded earlier that this value is unreasonably large and causes a dramatic increase in simulated peak breach outflow.

Embankment width and tailwater geometry

Ahn (2011) maintains that the FSAR and SER runs of the NWS-BREACH model were unduly constrained by limited tailwater section width and embankment length parameters provided as input to NWS-BREACH. PNNL investigated the sensitivity of NWS-BREACH results to the embankment length and found as expected that the results are only sensitive to the embankment length when the predicted breach width begins to exceed the specified embankment length. These tests were carried out with n values of 0.050 and 0.060. Ahn tested the sensitivity to embankment length using n=0.080 (Ahn 2011c) and n=0.075 (Ahn 2011b, Table 6.4-5) and with a larger tailwater cross section, and thus obtained larger peak outflows and breach widths, but there was no sensitivity to embankment length when it exceeded the predicted breach width. The primary source of the increased breach widths and peak outflows was the increase in the n value, and secondarily the increase in tailwater channel size. Since PNNL ultimately used n=0.075 in their final runs, they should have also tested the sensitivity at this n value. However, without also increasing the tailwater section, they would have still concluded that an embankment length of 1000 ft did not constrain the resulting breach width. Furthermore, this reviewer believes that values of n greater than 0.050 are inappropriately large; with n=0.050 or lower, the embankment length of 1000 ft used by PNNL would have been adequate to avoid constraining the breach width, even with a larger tailwater cross section.

Ahn (2011) also demonstrated sensitivity to the tailwater cross section size, but again did so with runs that utilized inappropriately large Manning's n values for the breach channel (n=0.075 in Ahn 2011b, Table 6.4-6; n=0.070 in Ahn 2011c). The larger breach widths and peak outflows were mostly the result of the increased n values. PNNL's sensitivity testing of this parameter again took place in runs that utilized n=0.050. In hindsight, since PNNL chose to use n=0.075 in their final runs, they should have tested the sensitivity to tailwater section size using this same value of n. However, this reviewer believes that n=0.050 is already an extremely conservative value and should have been used for the final analysis. At this value of n, there is minor sensitivity to the tailwater section size.

In reality, the NWS-BREACH model is poorly equipped to model the tailwater conditions for this situation, since outflow from the MCR embankment breach would immediately enter a relatively flat plain, instead of a defined river channel. Two-dimensional modeling is really needed to properly simulate the tailwater conditions in this area. Since NWS-BREACH is incapable of 2D modeling of the tailwater, the tailwater cross section should be sized to represent a realistic flow spread. The tailwater section used by PNNL (600-ft base width at El. 29 ft, widening to 2800 ft at El. 50 ft) seems appropriate, while Ahn's proposed 3000-ft wide rectangular tailwater section will produce unrealistically low tailwater levels by immediately forcing the flow to be spread thinly over a 3000-ft wide section.

Comparison to Martin Cooling Pond Breach

Ahn (2011a) objects to the use of the 1979 Martin Cooling Pond failure as a point of comparison to the MCR embankment breach because "...this process is highly subjective compared to the statistical regression methods that used multiple historical data points to come up [with] the best-fitting (or bounding) regression line of breach parameters." Ahn (2011b) also points out that the Martin Cooling Pond breach width (about 610 ft) was greater than the predicted breach width of the MCR embankment (417 ft) from the Froehlich (1995b) equation, and less than the breach width of several other dam failures reported in Wahl (1998) that might have been better candidates for comparison to the MCR embankment.

After reviewing the Wahl (1998) database, it appears that the Martin Cooling Pond is the most similar case considering embankment height, storage, and mode of failure. The Martin Cooling Pond was 28 ft high, impounded about 110,000 ac-ft, and failed by internal erosion. Although the observed breach width is somewhat greater than that predicted for the MCR embankment by the Froehlich (1995b) equation, the observed peak outflow is lower than that simulated for the MCR embankment with FLDWAV using the Froehlich (1995b) breach width and MLM (1984) breach time.

The typical purpose for a comparative analysis to historic dam failure cases is to confirm that results obtained by other methods (i.e., the regression-based breach parameters and confirmatory NWS-BREACH analysis) are reasonable, and the comparison achieves that. As Ahn (2011a) points out, the process is highly subjective. Unconsidered factors, such as different embankment materials and erodibility, could easily explain the difference in the results. With the information available, the regression-based analysis and NWS-BREACH analysis have the best potential to make an accurate prediction of the MCR breach characteristics, and the comparison to the Martin Cooling Pond failure is reasonable.

To learn more about the Martin Cooling Pond failure, this reviewer contacted Mr. Les Bromwell (les.bromwell@amec.com) of AMEC Corporation. Mr. Bromwell was a member of one team that investigated the failure. He reported that the embankment was constructed from non-plastic materials, both sand and silty sand, with a soil-cement armoring that did not significantly influence the size of the breach. These materials would be expected to exhibit high erodibility, which would lead to a rapid breach development and large breach size, with an associated large peak breach outflow. In contrast, the materials in the MCR embankment (clays with liquid limit greater than 30 or 40 depending on zone), should have dramatically greater erosion resistance, which would translate into a much slower breach, smaller breach width, and lower peak outflow. The contrast between materials in the Martin Cooling Pond embankment and the MCR embankment is similar to the contrast in materials used for laboratory-scale embankment overtopping breach tests by Hanson et al. (2005a). Those tests demonstrated differences in rates of headcut advance and breach widening of three orders of magnitude between embankments constructed from non-plastic silty sand (SM) and clay (CL, liquid limit=34) soils. Although not yet published, subsequent tests of embankment breach triggered by internal erosion have demonstrated similar sensitivity to clay content, plasticity, and compaction conditions (personal communication, Greg Hanson, USDA-ARS, Stillwater, OK).

Combined hurricane storm surge and MCR embankment breach

Ahn (2011a) raised questions about the selection of probable maximum hurricane (PMH) parameters by the applicant and the modeling of storm surge levels using the ADCIRC and SLOSH models. While other reviewers are addressing the technical issues related to hurricane and storm surge modeling, this reviewer was asked to comment on the potential for an MCR embankment breach caused by storm surge levels and currents that might attack the MCR northern embankment. For this purpose, the presently available predictions of hurricane and storm surge parameters and resulting water levels and currents are accepted as valid. Discussion of the issues related to a potential breach of the MCR embankment due to hurricane and storm surge effects is primarily contained in SER sections 2.4S.5.4.5, and 2.4S.10.4.

Figure 3 shows a cross section of the MCR embankment and a typical Texas City levee that survived several hours of sustained surge and wave attack during Hurricane Ike without a breach. ADCIRC simulations predicted probable maximum storm surge (PMSS) wave runup to elevation 29.3 ft MSL, which is below the 34 ft MSL grade elevation of the northern face of the northern embankment. At this level, water would be against the 6:1 toe berm. The toe of the main body of the embankment would be about 60 ft away. Even at a rate of several feet per hour, many hours would be required for erosion to begin to reach the main embankment. This loading seems incapable of causing an MCR embankment breach.

A SLOSH model described as very conservative predicted storm surge water surface elevation of 38.5 ft MSL and wind-wave action up to 41.8 ft MSL. In this event, water levels would be above 34 ft MSL (i.e., at site grade) for 80 minutes;

at or above 36 ft MSL for 50 minutes; and at or above 38 ft MSL for 25 minutes. Only this last elevation exceeds the level of the berm inspection road shown at 36.5 ft MSL, The northern face of the MCR northern embankment is grass-covered. Unless the currents and associated shear stresses against the embankment face are so high as to cause sod stripping or complete destruction of the vegetation, this loading seems incapable of causing erosion at a rate sufficient to produce an MCR embankment breach in the short duration of this loading. The size of the MCR embankment in comparison to the Texas City levee shown in Figure 3 also suggests that it would be in no danger of a PMSS-induced breach.

Water flow along the face of the MCR northern embankment was estimated by the applicant for the conservative SLOSH-modeled scenario using three different methods. Values of 11.6 ft/s, 3.1 ft/s, and 6.2 to 13.2 ft/s were obtained. Flow along the embankment would occur for up to 80 minutes. For this duration, Hewlett et al. (1987) recommend that grass-lined channels can sustain velocities of 9 to 14 ft/s, depending on the quality of the grass cover. The predicted velocities are comparable, suggesting that the grass cover would be able to withstand this level of hydraulic attack, although we are not told specifically about the quality and uniformity of the vegetation or the maintenance program for it. Even if the grass cover were damaged within this time frame, the clay content of the underlying zone B materials (clay with liquid limit ≥ 30) suggests that these materials would have at least moderate erosion resistance. Thus, it seems very unlikely that subsequent damage to the underlying embankment materials could be sufficient in this time period to lead to an MCR embankment breach.

The NRC staff considered only the loading predicted from the ADCIRC model and concluded that it was unlikely that the predicted water levels could lead to erosion of the embankment toe sufficient to form a pipe extending to the interior face of the embankment, or a sliding surface extending from the toe to near the upstream face of the embankment. Thus, they concluded that an MCR embankment breach due to PMSS loading was unlikely.

After reviewing the available information, this reviewer concurs that an MCR embankment breach due to probable maximum storm surge loading is not a credible failure mode. The presumed loading primarily attacks the toe berm and not the main body of the embankment; the duration of loading is short; the vegetal cover on the embankment is likely to prevent erosion into the embankment during the expect loading period; and the embankment soils themselves are expected to be at least moderately erosion resistant.

Conclusions

Table 2 provides a tabular summary of the findings of this review related to specific issues raised in the non-concurrence. The analysis based on breach parameter regression equations for breach width and time of failure and a subsequent FLDWAV analysis to predict peak outflow is technically sound. The chosen breach width equation (Froehlich 1995b) provided the smallest mean prediction error and uncertainty in a comparative study by Wahl (2004). The chosen failure time equation (MLM 1984) gave a shorter failure time than a companion failure time equation from Froehlich (1995b). Ahn's development of a best-fit failure time equation using data from MLM (1984) produces a slower breach with lower peak outflow, since the MLM (1984) equation estimated the lower envelope of failure times. With these conservative estimates of breach width and failure time, the resulting peak outflow from the FLDWAV model is very conservative. Comparison to a well-regarded peak breach outflow equation by Froehlich (1995a) confirms that the result is conservative.

The confirmatory analysis using the NWS-BREACH model verifies that the FLDWAV analysis is reasonable. All investigators confirmed that the model is very sensitive to the Manning's *n* value of the breach channel. Due to confusion about the proper form of the Strickler equation from typographical errors in some versions of the NWS-BREACH documentation, all investigators ultimately used extremely conservative values of Manning's *n*, which produced large, extremely conservative estimates of the peak breach outflow. These estimated peak outflows are comparable to and still do not exceed the results of the FLDWAV and regression-equation based analysis. With more realistic *n* values, the NWS-BREACH predictions of peak outflow would be much lower.

Specification of the tailwater channel size and embankment length can have some influence on NWS-BREACH results, but the effects are relatively small in comparison to the effects of Manning's *n* values. With reasonable values of Manning's *n*, the tailwater channel and embankment length values used in the analyses by the applicant and PNNL do not constrain the results, so the use of larger values would not alter study conclusions.

Effects of a scour hole that would form during a breach are uncertain. The physical mechanism by which a scour hole would increase discharge is poorly understood. Levee failures with large breach widths and high peak outflows have exhibited scour holes, but there is not conclusive evidence of a cause and effect relationship between scour holes and large breach outflows. The methods by which Ahn (2011b) incorporated scour holes into the NWS-BREACH model were not clearly explained and may not have adequately considered all important downstream factors. As a result, these analyses cannot be given credence.

The comparison of the MCR embankment breach to the Martin Cooling Pond failure is appropriate and confirms that the other analysis results are reasonable. The materials in the Martin Cooling Pond embankment were much more erodible than materials contained in the MCR embankment, so the potential breach of the MCR embankment should be much less severe than the Martin Cooling Pond failure.

The applicant, PNNL, and NRC staff have all concluded in the FSAR and SER that an MCR embankment breach caused by storm surge effects is not a credible failure mode. The reviewer concurs with that finding. The presumed storm surge loading primarily attacks the toe berm and not the main body of the embankment; the duration of loading is short; the vegetal cover on the embankment is likely to prevent erosion into the embankment during the expect loading period; and the embankment soils themselves are expected to be at least moderately erosion resistant.

In summary, the reviewer finds the MCR embankment breach analysis by the applicant, PNNL, and NRC staff as documented in the FSAR and SER to be technically adequate, conservative, and appropriate for establishing the design basis flood for safety related facilities associated with the expansion of the South Texas Project Electric Generating Station.

Table 2. — Summary of findings.

Issue	Finding
Incorrect choice of empirical breach parameter regression equations.	1) Froehlich (1995b) breach width equation is applicable to breach widths greater than 164 ft. 2) Prediction of breach width using MLM (1984) carries much greater uncertainty than Froehlich (1995b). 3) Use of MLM (1984) breach time equation is more conservative than using Froehlich (1995b) equations to predict all breach parameters
Incorrect application of NWS-BREACH model for confirmatory analysis.	NWS-BREACH model was applied in a conservative manner and confirms that results of analysis based on breach parameter equations and the FLDWAV model are reasonable.
Manning's *n* roughness value for breach channel	Value of n=0.025 is conservative and reasonable. Value of n=0.050 is extremely conservative. Use of n=0.075 or larger is not credible.
Effects of soil-cement blocks and sand core on Manning's *n*	Use of Chow (1959) procedure to incorporate effects of obstructions, vegetation, channel variability and other factors is inappropriate. The Strickler equation as given in original NWS-BREACH documentation or other methods that estimate Manning's *n* based on materials forming breach channel should be used.
Effect of scour hole	Physical mechanism by which scour hole will increase breach flow is not demonstrated, and method by which scour hole was incorporated into NWS-BREACH model was not documented. Effects of scour hole are unproven.
Embankment width and tailwater channel configuration	With reasonable selection of Manning's *n*, sensitivity to embankment length and tailwater channel size are insufficient to produce peak outflows that would change FSAR and SER conclusions.
Comparison to Martin Cooling Pond breach	Comparison is appropriate and confirms that results of other analyses are reasonable.
Potential for hurricane storm surge and combined MCR embankment breach	This failure mode is not deemed to be credible. The presumed loading primarily attacks the toe berm and not the main body of the embankment; the duration of loading is short; the vegetal cover on the embankment is likely to prevent erosion into the embankment during the expect loading period; and the embankment soils themselves are expected to be at least moderately erosion resistant.

References

Ahn, H., 2011a. South Texas Project Combined License Application Review: SER with no Open Item Chapter 2.4. June 8, 2011. 4 pp.

Ahn, H., 2011b. Enclosure 1: Re-Analysis of MCR Breach Flood, June 20, 2011. 54 pp.

Ahn, H., 2011c. Enclosure 2: PNNL's Calculation Package – Commented and corrected by Dr. Hosung Ahn. 13 pp.

Chow, V.T., 1959. *Open-Channel Hydraulics*. McGraw-Hill Book Co., New York, 680 p.

Fread, D.L., 1977, "The Development and Testing of a Dam-Break Flood Forecasting Model," Proceedings of the Dam-Break Flood Routing Model Workshop, Bethesda, MD, p. 164-197.

Fread, D.L., 1984, DAMBRK: The NWS Dam-Break Flood Forecasting Model, National Weather Service, Office of Hydrology, Silver Spring, Maryland.

Fread, D.L., 1988 (revised 1991), BREACH: An Erosion Model for Earthen Dam Failures, National Weather Service, National Oceanic and Atmospheric Administration, Silver Spring, Maryland.

Fread, D.L., 1993, "NWS FLDWAV Model: The Replacement of DAMBRK for Dam-Break Flood Prediction," Dam Safety '93, Proceedings of the 10th Annual ASDSO Conference, Kansas City, Missouri, September 26-29, 1993, p. 177-184.

Fread, D.L., and J.M. Lewis, 1998. NWS FLDWAV Model: Theoretical Description and User Documentation, Hydrologic Research Laboratory, Office of Hydrology, National Weather Service, Silver Spring, Maryland, November 28, 1998.

Froehlich, David C., 1995a. Peak outflow from breached embankment dam. *Journal of Water Resources Planning and Management*, 121(1):90-97.

Froehlich, David C., 1995b. Embankment dam breach parameters revisited. In Proceedings of the 1995 ASCE Conference on Water Resources Engineering, San Antonio, Texas, August 14-18, 1995, p. 887-891.

Hanson, G. J., K. R. Cook, and S. L. Hunt, 2005a. Physical modeling of overtopping erosion and breach formation of cohesive embankments. *Transactions of the ASAE*, 48(5): 1783−1794.

Hanson, G. J., D. M. Temple, M. Morris, M. Hassan, and K. Cook. 2005b. Simplified breach analysis model for homogeneous embankments: Part II, Parameter inputs and variable scale model comparisons. Proceedings of 2005 U.S. Society on Dams Annual Meeting and Conference, Salt Lake City, Utah. p. 163-174.

Hanson, G.J., Temple, D.M., Hunt, S.L., and Tejral R.D., 2011. Development and characterization of soil material parameters for embankment breach. *Applied Engineering in Agriculture* (accepted for publication on March 22, 2011).

Hewlett, H.W.M., Boorman, L.A., and Bramley, M.E., 1987. Design of reinforced grass waterways. CIRIA Report 116, Construction Industry Research and Information Association, London, 116 pp. (see Figure 9).

MacDonald, T. C., and Langridge-Monopolis, J. 1984. Breaching characteristics of dam failures. Journal of Hydraulic Engineering, 110(5), 567–586.

Mohamed, M.A.A., 2002. Embankment breach formation and modelling methods. Ph. D. thesis. Open University, UK.

Pacific Northwest National Laboratory, 2011a. Response To: Re-Analysis of MCR Dam Breach Flood (June 20, 2011).

Pacific Northwest National Laboratory, 2011b. Electronically marked-up PDF copy of Ahn (2011b).

Smart, Graeme M., 1984. Sediment transport formula for steep channels. *Journal of Hydraulic Engineering*, 110(3):267-276.

South Texas Project, undated. South Texas Project Electric Generating Station Updated Final Safety Analysis Report (STPEGS UFSAR).

Special Board of Consultants, 1980. *Study of Cooling Pond Dike Failure, Martin Power Plant*, July 1980. Prepared for Florida Power and Light Company, Miami, FL.

Tejral, R.D., G.J. Hanson, and D.M Temple, 2009. Comparison of two process based earthen dam failure computation models. In *Dam Safety 2009*, Annual Meeting of the Association of State Dam Safety Officials (ASDSO), Sept. 27-Oct. 1, 2009, Hollywood, FL.

Temple, D. M., G. J. Hanson, M. L. Neilsen, and K. R. Cook. 2005. Simplified breach analysis model for homogeneous embankments: Part I, Background and model components. Proceedings of the 2005 U.S. Society on Dams Annual Meeting and Conference, Salt Lake City, Utah. p. 151-161.

Wahl, Tony L., 1997. Predicting embankment dam breach parameters - a needs assessment," 27th IAHR Congress, International Association for Hydraulic Research, San Francisco, California, August 10-15, 1997.

Wahl, T.L., 1998. *Prediction of Embankment Dam Breach Parameters: A Literature Review and Needs Assessment*, Dam Safety Office Research Report DSO-98-004, U.S. Department of the Interior, Bureau of Reclamation.

Wahl, T.L. 2004. Uncertainty of predictions of embankment dam breach parameters. *Journal of Hydraulic Engineering*, 130(5): 389-397.

Wahl, T.L., 1997, Predicting embankment dam breach parameters – a needs assessment, XXVIIth IAHR Congress, San Francisco, CA.

Wahl, Tony L., Gregory J. Hanson, Jean-Robert Courivaud, Mark W. Morris, René Kahawita, Jeffrey T. McClenathan, and D. Michael Gee, 2008. Development of next-generation embankment dam breach models. U.S. Society on Dams, 2008 Annual Meeting and Conference, April 28-May 2, Portland, Oregon,

Yen, B.C., 1991. Hydraulic resistance in open channels. In *Channel Flow: Centennial of Manning's Formula*, B.C. Yen, editor. Water Resources Publications, Littleton, CO., p. 1-135.

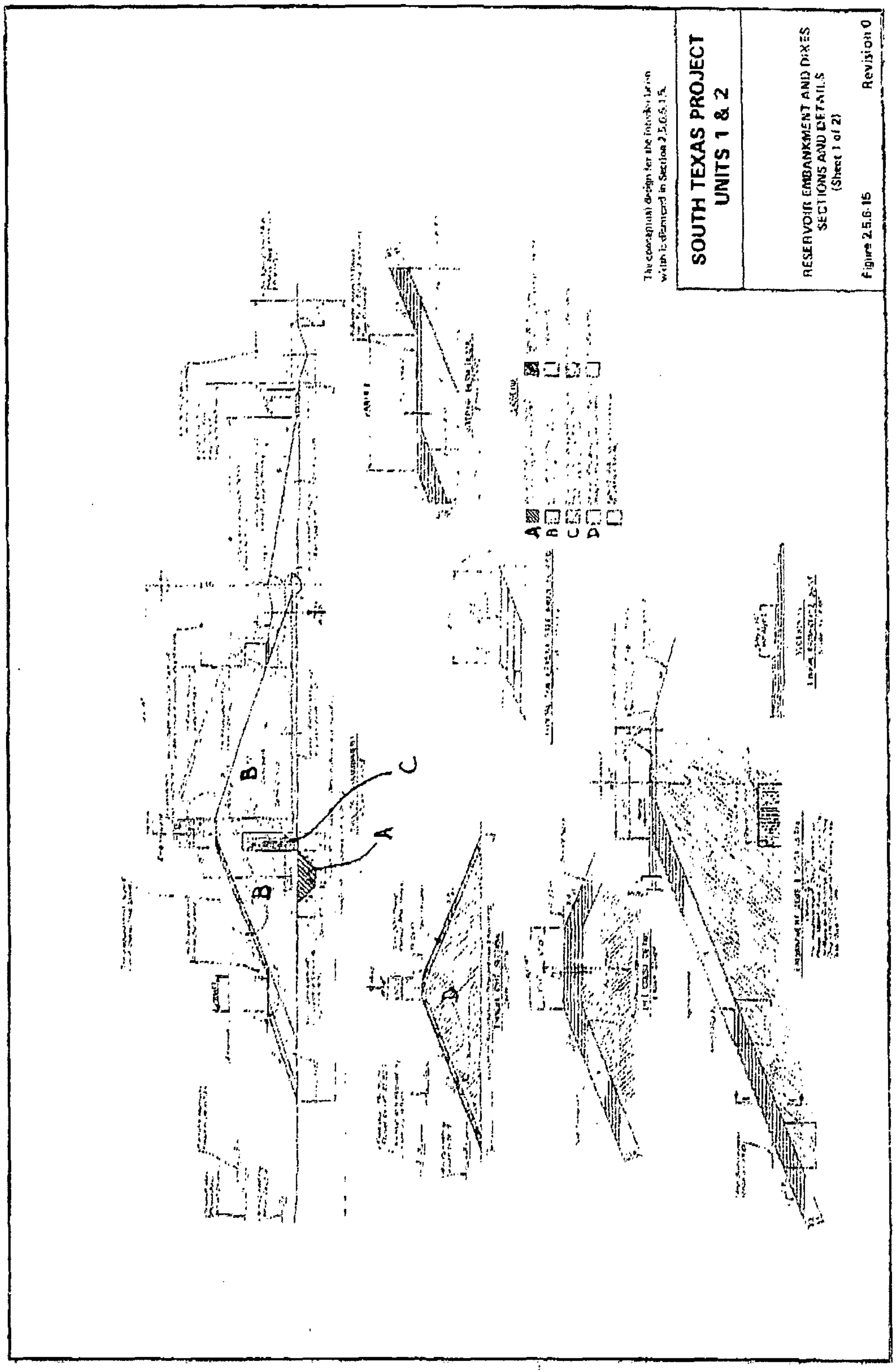

Figure 1. — MCR embankment sections showing zoning.

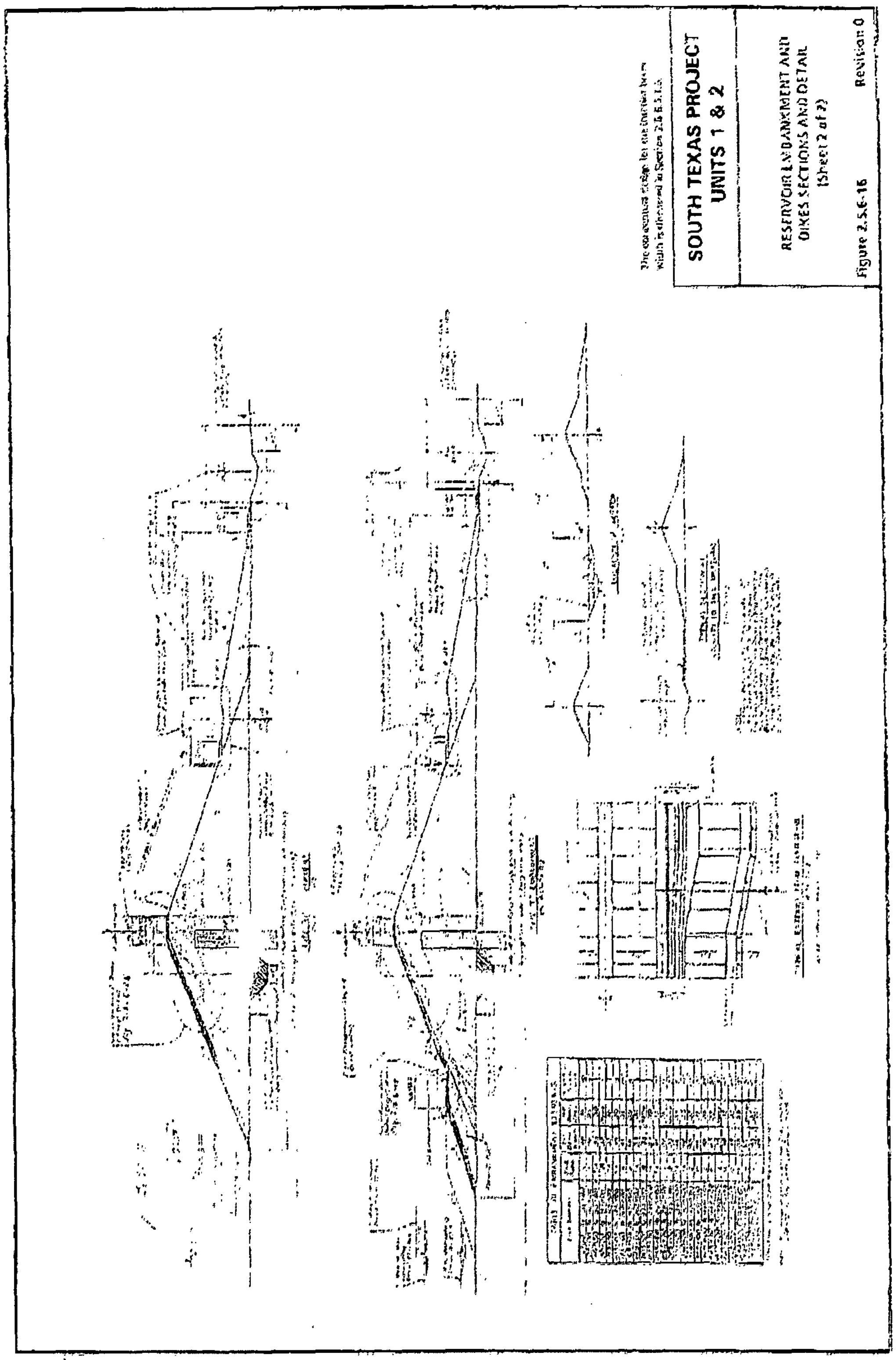

Figure 2. — Embankment sections, sheet 2 of 2.

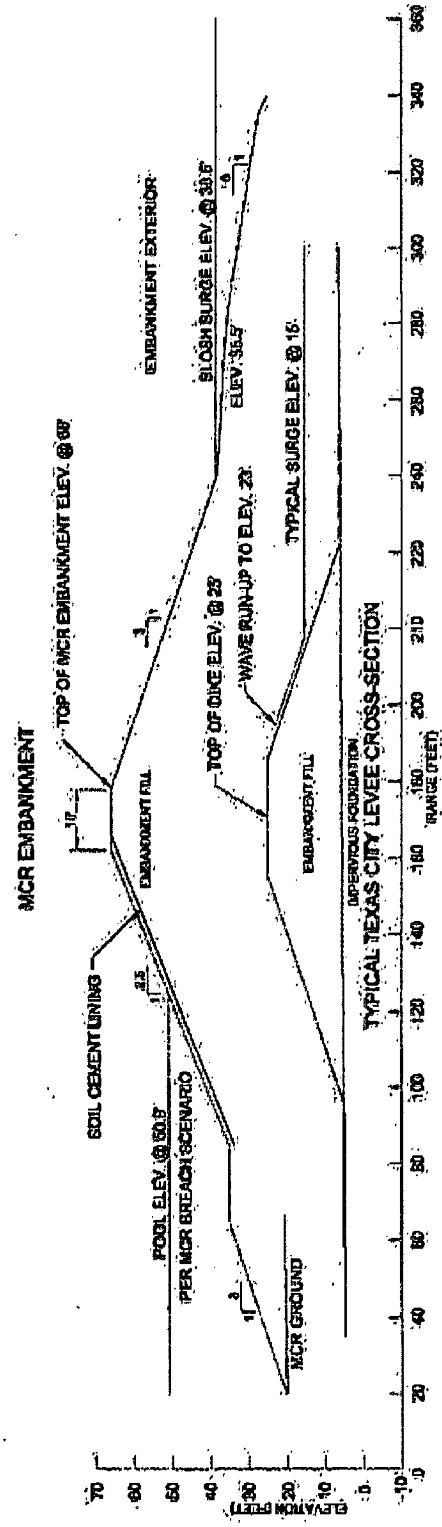

Figure 3. — Cross section of northern MCR embankment and a typical Texas City levee.

Attachment 8: STP NC Review MCR Breach – Patev November 30, 2011

Independent Technical Review for the Main Cooling Reservoir

Embankment at the South Texas Project Units 3&4

Independent Technical Reviewer:

Robert C. Patev, Senior Risk Advisor, Risk Management Center, US Army Corps of Engineers

Technical Review

1. Background

The reviewer was requested to examine and provide his technical expertise on the breach potential and breach characteristics of the Main Cooling Reservoir (MCR) embankment at the South Texas Project Electric Generating Station (STPEGS) situated near Corpus Christi, Texas. This review is based on a collection of review documents provided by the Nuclear Regulatory Commission (NRC) technical staff as well as external publications that the reviewer has utilized to assist in offering his opinion on the breaching potential and breach size of the MCR embankment. The review will focus on the geotechnical aspects of a failure along the northern section of the MCR embankment that would create a breach and inundate the proposed Units 3 and 4 at STPEGS. The review was conducted independent of the other technical reviewers and is the sole opinion of the reviewer based on the facts and information presented by the NRC. This opinion is the technical opinion of the reviewer and not the position of the reviewer's agency or the Department of Defense.

2. Understanding/Comparison of facts and findings presented to reviewer

The reviewer has extensively reviewed the Pacific Northwest National Laboratory (PNNL) report and calculations worksheet, the NRC Re-Analysis of MCR Breach Flood and PNNL's response to the NCR Re-Analysis. All these documents are very well written and provide a significant examination of the literature devoted to breach models and computer programs, input model parameters, and breach width and depth characteristics. Unfortunately, each party (now referred to as authors) has offered widely differing opinions in terms of MCR structure definitions, breach models/parameters and breach widths. The reviewer will try to synthesize their differences and offer his opinion based on their findings and critique of each others' opinions.

Topic	PNLL	NCR Re-Analysis
Structure	Embankment Dam	Levee
Structure Failure Mode	Seepage through embankment	Overtopping of Levee
Breach time	McDonald and Langridge (MLM) 1.7 hours	USBR, MLM, Von Thun and Gillette and Froehlich 0.66, 2.54, 2.68 and 7.98 hours respectively
Breach width	Froehlich 417 feet Limited – 600 feet @ tailwater And 1000 feet @ crest	USBR, MLM, Von Thun and Gillette and Froehlich 66, 1736, 235 and 417 feet respectively MLM – Table 6.3.2 Scour 0 feet – 1736 Scour = 10 feet – 1047 feet Scour = 20 feet = 745 feet
Breach model	NWS BREACH => FLDWAV => RMA2	1. NWS BREACH => FLO-2D 2. EMP EQNS => FLDWAV => FLO-2D
Breach flow	62,600 cfs	USBR, MLM, Von Thun and Gillette and Froehlich 22,607, 217,000, N/A, 60,078 cfs respectively
Manning Coefficient	0.075	0.085 (MLM)
Scour of foundation	None Assumed bottom = 29 ft MSL	Included 0, 10, and 20 feet

Table 1. Comparison Table of Information Provided by Authors

In addition to these comparisons, this review will focus only on the natural hazards applicable to the MCR embankment that are analyzed in both the PNNL submittal and the NRC Non-Concurrence analysis and documentation. This review will assume that the storm surge elevation predictions are reasonable and credible, but with the understanding that they are highly uncertain depending upon which models and parameters that are utilized in the surge and wave analysis.

3. Technical discussion of geotechnical design and subsurface information for MCR embankment

The authors never fully discuss, in their own opinions, the details as to the design, materials, construction methods, and seepage control used in the MCR embankment. These are critical factors and drivers, in the reviewer's opinion, that directly affect the failure mode (i.e., initiation, progression, continuation, and breach) for the embankment and affect the final breach opening parameters and potential scouring of the foundation.

One of the significant points of disagreements between the authors is the consideration of what type of structure has been constructed. Both authors feel the "type" of structure is very important when considering the type of breach that can occur at embankment or levee MCR embankment. This is based in part of how they want to present the historical data for examining breach widths.

This may be somewhat true due to differences in loading conditions (short versus long-term), however, the reviewer states that regardless of whether the structure is considered an embankment dam or levee, the construction for seepage control is basically the same for both. The MCR structure has been designed using compacted layers of silt-clay and fat clay (CL-CH) with sand (SM-SP) drains (chimney (Type A) and toe (Type B)). All the materials were excavated from local borrow areas near the STP site. The system has been designed with a significant seepage control system using a series of relief wells, piezometer lines and berms (interior and exterior) for stability. These design concepts are very similar to those detailed in the USACE Engineering and Design Manuals for Seepage Analysis and Control Dams (USACE, 1993) and the Design and Construction of Levees (USACE, 2000). Hence, for simplicity, the reviewer will only use the term "embankment" since the term is defined in USACE (2011) as "A raised structure of earth, rocks, or gravel, usually intended to retain water or carry a roadway".

Another factor that is important is that the embankment is composed of compacted silt-clay (CL-CH) material from a local borrow area. This material has a low permeability in the order of 10^{-5} or 10^{-6} cm/sec. Therefore, a seepage failure is highly unlikely given it has been in operation for a number of years. In addition, the foundation materials consist primarily of two different high strength clays which will assist with foundation erosion during a breach of the embankment.

Section 2.5.6.6 in the UFSAR details the seepage control system in place at the MCR embankment and Section 2.5.6.11 details the specifications to the seepage control system under loading to the current essential control pond (ECP) water elevations that are present today. There have also been additions to the relief well system and additional piezometer lines that were installed as the equilibrium of the seepage through the embankment occurred. The system has been working well and is properly designed. Overall, the STP MCR embankment is a properly designed and functioning structure. However, reexamination of the seepage control system will arise once the ECP water levels are raised to El 50 for Units 3 and 4.

4. Technical discussion of potential failure modes for MCR embankment

According to the design documentation that was reviewed, the MCR embankment has been designed to adequately address the natural hazards that are present at the STP site. The usual hazards for the MCR embankment are seepage, both through and under the structure, and slope stability. The unusual hazards are storm surge (wind and waves) and flooding due to a hurricane or subtropical event, or liquefaction from seismic events. The reviewer will not address the slope stability or seismic loads as part of this review even though they are mentioned in the authors' reports. However, the potential failure modes (USACE, FEMA) for the MCR structure that are considered plausible by the reviewer are:

a. Overtopping of embankment causing breach

The likelihood of a breach caused by overtopping of the MCR embankment is highly improbable even with significant rainfall and winds during design basis flood (DBF) or probable maximum hurricane (PMH) event. The main reason is that approximately 15 feet of freeboard will be present when Units 3 and 4 are in full operation and the waves and wind surge across the essential cooling pond (ECP) could not overtop the embankment. Even if they were able to overtop the embankment, the grass surface on the side and the concrete inspection roads on the down slope would assist in preventing any erosion on the backside of the embankment. If a more detailed PFMA was conducted, it would rate this failure mode as unlikely and it would be removed from further analysis in any additional studies.

b. Erosion of soil-cement barrier causing breach

The likelihood of a breach caused by erosion of the 2-ft thick soil-cement barrier on the MCR embankment and exposure to the compacted clays with additional headwater is highly improbable during design basis flood (DBF) or probable maximum hurricane (PMH) event. The maximum waves across the ECP would not have the energy to fully damage the soil-cement barrier to cause this condition to occur. If a more detailed PFMA was conducted, it would rate this failure mode as unlikely and it would be removed from further analysis in any additional studies.

c. Overtopping from storm surge

From the PMH analyses (i.e, SLOSH or ADCIRC) data presented to the reviewer, the maximum exterior surge and waves will only reach approximately El 36 to 38 feet

MSL and will be significantly short of overtopping the embankment. However, some minor damage on the upslope and top of the stability berm might occur but not to significant levels to consider exposure of new seepage paths. Even if storm surge was significantly higher due to uncertainties in the analyses, the embankment is covered with stable grass on the exterior and a soil-cement surface on the interior. If a more detailed PFMA was conducted, it would rate this failure mode as unlikely and it would be removed from further analysis in any additional studies.

d. Seepage and Piping through MCR Embankment

The MCR embankment is constructed with seepage control using both sand drains and relief wells. Given that the embankment is constructed primarily of compacted clay (CL-CH), the major seepage paths would have been sealed during the first fill of the ECP and any highly dispersive clay would pipe into the sand drains. Since there has been little to no evidence of continual seepage problems, no settlement of the embankment and with the additional installation of relief wells and piezometers, it appears the seepage and piping under the current state has reach water balance equilibrium. However, significant increases in water elevations during the DBF or PMH could create instability of the equilibrium of the current seepage conditions. This is would be the primary mode of failure of this embankment If a more detailed PFMA was conducted, it would state that is this is a credible and significant failure mode during any substantial change in water elevations in the ECP.

e. Seepage and Piping under MCR Embankment

The potential for seepage and piping under the embankment due to the sand layers (SM-SP) that exist below the clay lining surface of the ECP and MCR embankment is a credible failure mode. However, seepage control measures defined in UFSAR 2.5.6.6.1.1 reduce this likelihood greatly. Much like the seepage through the embankment, this potential failure mode does not appear likely, given there are no reports of significant water discharges or boils around the MCR embankment. However, significant increases in water elevations during the DBF or PMH could create instability of the equilibrium of the current seepage conditions. This would be the secondary mode of failure of this embankment. If a more detailed PFMA was conducted, it would state that is this is a credible and likely failure mode during any substantial change in water elevations in the ECP.

5. Technical discussion of breaching analysis of MCR embankment

The empirical breaching equations and computer programs discussed by the authors are widely used and accepted in engineering practice throughout the world. These models are interpretations for a wide variety of breach conditions (i.e., varying differential heads, embankment materials, foundation materials, embankment construction, failure modes, etc....) and contain large model uncertainties both aleatory and epistemic. So, is there one correct answer for the MCR embankment? As stated in the NRC non-concurrence, "Both USBR (1998) and USACE (2007) recommend using all available methods to determine breach parameters as the process involves a lot of uncertainty" so therefore, as the state-of-the-practice goes, the answer is still no. To the reviewer, the predictions of dam breach parameters is more an art than a science but given what has been presented by the authors, the reviewer will try to give his best opinion on the best estimate and range for the MCR embankment based on these differing analyses.

First, the inclusion of a scour hole into this breach analysis is not recommended due to the underlying foundation soils. The embankment is founded on clay stratum (1a) that is moderately plastic (CL-CH) and is considered moderately stiff with undrained cohesive shear strength in excess of 1000 psf (UFSAR 2.5.6.2.1.1). Below this is another clay stratum (1b) that is very stiff and has undrained shear strengths in excess of 2000 psf (UFSAR 2.5.6.2.1.1). Given these soil conditions, it would make it most unlikely to see erosion in the embankment foundation. Similar conditions were seen in the hurricane protection levees around New Orleans during Katrina where the enlargement hydraulic fills eroded easily and the stiff (CL-CH) clays in the foundation did not erode significantly during breaching (IPET 2008, Briaud 2008).

Second, the primary failure mode of the MCR embankment is the initiation, progression, continuation and breach due to a piping failure. This loss of containment and breach would more resemble that of a failed dam than that of a failed levee section. This is because the MCR reservoir would breach and lose its containment very quickly. Riverine levee breaches are different in that the erosion of the levee breach occurs over a much longer period of time than a dam breach since there is infinite water and hence have longer breach widths and scour potential. Eventually, a levee breach has to reach equilibrium to balance river water elevation with the protected side water elevation. Then reverse flow and some levee erosion occurs as the river stage comes down. This is not the case for the MCR embankment.

Third, the majority of the material found in the embankment is compacted silty clay (CL-CH) that has an effective strength of approximately 300 psf. These would be considered a medium to low erodibility material as defined by Briaud (2008) and would have some resistance to flow through the breach depending upon duration. The sand drain materials would be considered a highly or very highly erodible material as defined by Briaud (2008) and would be removed quickly during a pipe and breach condition. However, the sand would not expedite the seepage path as the pipe has to intersect another layer of clay before it can exit the embankment. Additionally, the seepage path has been lengthened due to the presence of the stability berms on both the interior and exterior sides. Therefore, the breach would evolve slowly and then erode away most of the embankment materials down close to but above the foundation level.

Therefore, given this information and in the opinion of the reviewer, it would appear that using the Froehlich (1995) approach would lead to a reasonable estimate of the breach width of the MCR embankment. However, given the information presented in Wahl (2004), it would appear that the Froehlich method also gives the least amount of dispersion about the predicted versus observed breach width values. Therefore, assuming a breach width between 400 and 600 feet would be a reasonable estimate for the MCR embankment.

As for time of failure predictions, the MacDonald and Langridge-Monopolis (1984) method used by PNNL seems to give reasonable predictions as to the time to peak outflows for this type of embankment and materials that are present. The Froehlich (1995) approach seems too conservative given the materials and reservoir capacity at STP MCR.

6. Reviewer's Opinion and Recommendations

In summary, the reviewer feels that both authors have done an excellent job expressing their own viewpoints and merits to their approach to this problem. While they disagree on many fine-point technical issues, it appears that they both are concerned with the safety of the STP MCR embankment and the protection of Units 3 and 4 from DBF levels. Again, as stated earlier, there is no exact answer to this problem since the uncertainties in the equations, models and programs are just not well quantified, but a best estimate and range of breach width and times are warranted to resolve these differences in the results.

Therefore, it is the opinion of the reviewer, that the breach analysis conducted by PNNL does follow a solid overall approach and a consistent methodology required for the breach analysis for the MCR embankment. The NCR Re-analysis is also very beneficial since it does show a sensitivity analysis to the PNNL methodology that could be utilized to estimate the range of the breach widths and times to assist with the final decision of the permitting of Units 3 and 4 at STP.

References

Briaud, J.L., Chen H.C., Govindasamy, A.V. and Storesund, R. Levee Erosion by Overtopping in New Orleans during the Katrina Hurricane, Journal of Geotechnical and Geoenvironmental Engineering, Vol. 134, No. 5, May 2008, pp. 618-632

Froehlich, D. C. 1995. Embankment Dam Breach Parameters Revisited. Water Resources Engineering, Proceedings of the 1995 ASCE Conference on Water Resources Engineering, New York, 887-891.

IPET 2008. Performance Evaluation of the New Orleans and Southeast Louisiana Hurricane Protection System, Final Report of the Interagency Performance Evaluation Task Force (IPET) – Hurricane Katrina: Volume VIII – Operational Risk and Reliability.

MacDonald, T.C., and Langridge-Monopolis, J. 1984. Breaching Characteristics of Dam Failures. Journal of Hydraulic Engineering 110(5), 567-586.

USACE 1993. EM 1110-2-1901, Engineering and Design: Seepage Analysis and Control of Dams.

USACE 2000. EM 1110-2-1901, Engineering and Design: Design and Construction of Levees.

USACE 2011. ER-1110-2-1156, Engineering and Design; Safety of Dams -Policy and Procedure.

Wahl, T.L. 2004. Uncertainty of Predictions of Embankment Dam Breach Parameters. Journal of Hydrology, 130(5), 389-397.

Attachment 9: STP NC Review MCR Breach – Beacher

November 22, 2011

Memorandum

TO: Joseph Kanney, USNRC
FROM: Gregory Baecher, University of Maryland
SUBJECT: RES---11---267 "INDEPENDENT TECHNICAL REVIEW OF THE MAIN COOLING RESERVOIR DAM BREACH"
DATE: 7 November 2011

Referenced documents

1. Safety Evaluation Report (SER)
2. Final Safety Analysis Report (FSAR)
3. BREACH Verification and Validation Summary (FSAR)
4. Re-Analysis of MCR Breach Flood (Non-Concurrence)
5. Reasons for Non-Concurrence (Non-Concurrence)
6. Calculation Worksheet (Non-Concurrence)
7. Response To: Re-Analysis of MCR Breach Flood (Non-Concurrence PNNL Response)
8. Probable Maximum Storm Surge Review and Analysis Performed by Staff (Non-Con. PNNL Response)
9. UFSAR
10. MCR Geotechnical Report STP Units 1 and 2 (UFSAR) with drawings

Introduction

The objective of the review is to evaluate the technical adequacy of the of the Main Cooling Reservoir (MCR) embankment breach analyses. The breach analysis performed by the South Texas Nuclear Plant (STP) licensee was reviewed by Pacific Northwest National Laboratory (PNNL) and included in the STP Safety Evaluation Report (SER). A non-concurrence on the dam breach Safety Evaluation Report (SER) was filed by the Hydrologic Engineering Branch (HEB) and asserts that the flood emanating from the MCR embankment breach had been underestimated (Documents #4, 5, 6). Assessment of the analytical methodologies and models, and computer input parameter values used by PNNL have been reviewed to determine the validity of the breach analysis.

The question at hand is the probable flood at the reactor given that a dam breach occurs. The question of the probability of a breach within some interval of time is taken as irrelevant to the question.

Summary opinion

Both sides of the technical argument seem flawed, although in different ways.

- In my view, the NRC non-concurrence (#4) stretches its arguments in favor of the most extreme condition beyond the support of historical data on dam breaches and beyond what appears to be the actual geometry and loading patterns of the MCR. I do not think that the arguments of the NRC non-concurrence by themselves fully support the conclusion that the PNNL analysis is "neither accurate nor conservative."

- On the other hand, the PNNL analysis and response (#7) to the NRC non-concurrence seems to overstate its case, ignore significant statistical uncertainties, over-simplify potential loading events, and make a poor case for ignoring multiple failure modes.
- A major limitation of both the PNNL analysis and the USNRC non-concurrence is that they fail to adequately address significant epistemic uncertainties in the modeling.
- Given that the normal maximum operating pool in the MCR is 49 feet and that the minimum crest elevation of the MCR embankment is 65.75 feet, the probability of overtopping due to rainfall appears negligible. While I am not an expert in hurricane surge analysis, the estimated maximum surge based on what appear to be conservative assumptions, and including a reasonable allowance for sea level rise, does not appear to pose a credible threat of overtopping of the MCR, given the base elevation and height of the structure.

Thus, while the NRC non-concurrence fails to convince this reviewer that the PNNL analysis is "neither accurate nor conservative" (#4), at the same time the PNNL analysis seems inadequate for other reasons. Whether these other reasons are of sufficient importance and whether they have sufficient impact on the uncertainties attending the breach flood prediction to negate the conclusions of the PNNL analysis is outside the scope of this review, which precludes any re-analysis.

Technical issues

The issues separating the NRC non-concurrence and the PNNL analysis appear to hinge on the following:

- Whether the MCR "fails in the pattern of a dam or a levee," and
- Thus the character of the tailwater downstream of the breach, and thus the outflow hydrograph.

This, in turn, depends on,

- The failure modes considered, and
- The internal design of the MCR embankment.

Subsequently, the prediction of outflow and downstream flooding depend on,

- Choice of the best forecasting model of breach discharge, and
- Whether a scour hole forms at the toe of the breach.

An issue that is not identified either by the NRC non-concurrence (#4) or the PNNL analysis and response (#7) is the degree of uncertainty in predictions of breach discharge attending statistical regression analysis. This uncertainty is substantial and may cause a non-negligible increase in risk over that reported by PNNL.

The NRC non-concurrence (#4) bases its argument on an assumption that the MCR will behave as levees do when they fail, which is taken to mean, failing over a broad stretch and releasing high discharges over a broad downstream area of flooding. That this failure performance applies to the MCR seems, to me, not well supported. The non-concurrence also uses a statistical correlation of outflow discharge to reservoir characteristics that does not seem as good as the Froehlich correlation that the PNNL analysis is based on.

Yet, the PNNL analysis has flaws, too: (i) it ignores all but piping failure modes, (ii) it fails to account for large uncertainty in the Froehlich correlation, (iii) it dismisses the contention that tailwater conditions may be much different than assumed, and (iv) it dismisses the possibility of a scour hole.

Whether the MCR fails in the "pattern of a dam or pattern of a levee"

To avoid semantic argument over whether the MCR is a "dam" or a "levee," the term *embankment* is used.

The issue here, it seems to me, is the width of the breech through which outflow occurs and consequently the tailwater conditions inhibiting that outflow. PNNL argues that the width of the tailwater is narrowly confined; whereas, the NRC non-concurrence argues that the tailwater cross-section should be the width of the downstream valley. No one appears to deny the importance of this assumption.

- PNNL argues that a piping failure mode — the only one likely in PNNL's opinion — leads to a narrow outflow channel; this leads to higher tailwater, rapid transition to sub-critical flow, and lessened discharge.
- The NRC non-concurrence argues that on overtopping failure mode or an unraveling failure mode like those seen in levee systems, leads to a broad outflow sheet; this leads to lower tailwater, slow transition to subcritical flow, and heightened discharge.

Each of these assertions seems correct to me, given their respective presumptions of failure mode.

Failure modes and the internal design of the MCR embankment

Breach geometry depends on (i) the failure mode and (ii) the internal design of the embankment. The long discussions of the best way(s) to assign Manning's friction coefficient to the dam material and the downstream channel seem secondary to the main question, which is the geometry.

PNNL argues, not implausibly, that a piping-caused breach is likely to be narrow and deep. For a piping failure and narrow-deep breach, the tailwater will likely be as PNNL assumes, reducing the breach discharge.

If the embankment overtops over a wide extent, or if seismically induced liquefaction occurs over a large area, it seems possible that a wide breach could open and a broad outflow result. A liquefaction induced failure or overtopping would be likely to generate a wider and shallower breach. The resulting outflow hydrographs would, in principle, be quite different in the two cases.

Failure modes

Three failure modes are mentioned in the PNNL report for the MCR: (1) Seismic groundshaking leading to soil liquefaction, (2) overtopping due to high reservoir inflows, and (3) internal erosion in the embankment leading to piping. For reasons that seem poorly argued, only the last is quantitatively considered in the flood hazard analysis.

This is important because the characteristics of the resulting breach and discharge are affected by the failure mode. Yet, we know from historical experience that piping in earth dams is the cause of fewer than half of all loss-of-containment failures.[1] While rates vary among the relevant databases, about 40% of earth dam failures are due to piping, about another 40% are due to overtopping or spillway inadequacy, and the remaining 20% are due to slides or miscellaneous factors.[2] Relatively few historical dam failures have been caused by seismicity, but the number of dams exposed to seismic hazard is only a fraction of the total number of dams, and many dams today are thought to be susceptible to seismic damage.

Piping is caused by internal erosion which develops over time under an hydraulic seepage gradient in the dam. Such failures can occur in normal ('sunny day') operation as well as during first-filling or flood events. Overtopping failures typically occur during flood events, although they may occur at other times due to mis-operation or spillway system inadequacies. A dam failure triggered by high reservoir inflow during a hurricane would seem more likely to be caused by overtopping rather than piping.

Internal design of the embankment

Several things differentiate a levee from a dam, but most are irrelevant to the current question. Things that are relevant include: (1) Levees are minimally-engineered structures with little or no internal zonation, (2) they usually have water against them only sporadically, (3) they are usually built on a foundation of unimproved natural soils, and (4) they are usually extremely long. Only the last of these applies to the MCR embankment.

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[1] Foster, M., Fell, R., and Spannagle, M. (2000). "A method for assessing the relative likelihood of failure of embankment dams by piping," *Canadian Geotechnical Journal*, 37: 1025-1061.

[2] Baecher, G.B., Pate, M.-E, and deNeufville, R. (1980). "Dam failure in cost-benefit analysis," *Journal of the Geotechnical Engineering Division*, ASCE, *106*(GT1): 101-105

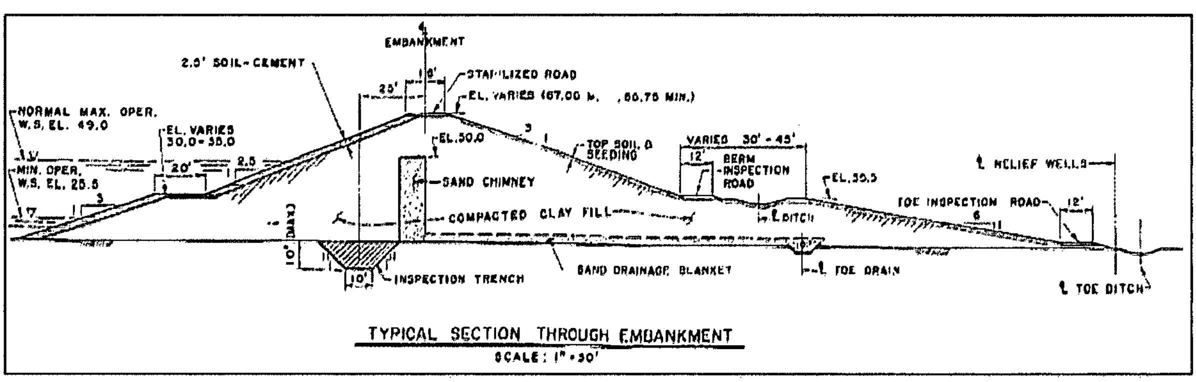
Considerations

PNNL argues that the Bureau of Reclamation's Dam Safety Office (USBR-DSO) database supports the claim that the MCR embankment will fail in a narrow-deep breach. This seems an inappropriate conclusion since the DSO's data base deals only (or predominantly) with dams that are distinctly different in their geometry, height, and possibly their failure modes from the MCR embankment.

The Martin Cooling Pond failure, while an interesting point of comparison, does not shed light on the likelihood of an extended breach due to overtopping. That failure is reported by Wahl[3] to be due to a foundation defect so presumably shares more characteristics with piping failure modes than with overtopping ones. The many other historical examples, while of great interest, similarly seem to differ enough from the MCR embankment as not to shed much light on the current site.

Will the embankment be overtopped and fail? PNNL posits that the embankment will not overtop (p.5 of #7) because there is insufficient reservoir inflow. If the hydrology-hurricane review substantiates the contention that the overtopping probability is diminishingly small, then this failure mode could be dispensed with, and the presumption of only deep-narrow breaching due to piping would be supported. But that conclusion is outside the scope of the present review.

The non-concurrence notes that breach lengths in riverine levee systems can be large. However, comparing the MCR embankment to the California Delta levees or to flood levees on the Danube is inappropriate. The California Delta levees and the Danube levees date from the 1800's and to a large extent did not benefit from engineering design or construction controls. They are mostly homogeneous rather than zoned fills. Many reaches in both systems are poorly maintained. In contrast, the MCR embankment is compacted clay fill with a 2.5 foot soil cement upstream facing (Figure 1). The condition of the grass cover on the downstream face is unknown, but if well-maintained would be a significant inhibitor of overtopping erosion.



TYPICAL SECTION THROUGH EMBANKMENT
SCALE: 1" = 30'

[3] Wahl T.L. (1998). "Prediction of Embankment Dam Breach Parameters: A Literature Review and Needs Assessment.," DSO-98-004, U. S. Bureau of Reclamation, Denver.

Figure 1. Profile of the MCR embankment (from USFAR #9)

My conclusion is that PNNL's argument rests in large measure on the presumption that an overtopping failure is not possible. If that is accepted, then the issue of failure mode seems moot, as does the issue of low tailwater and broad downstream flows. If that presumption is not accepted, then the more severe inundations of the non-concurrence have at least some credibility. However, the historical evidence for broad breaching in reservoir impoundments is sparse.

Given that the normal maximum operating pool in the MCR is 49 feet and that the minimum crest elevation of the MCR embankment is 65.75 feet, the probability of overtopping due to rainfall appears negligible. While I am not an expert in hurricane surge analysis, the estimated maximum surge based on what appear to be conservative assumptions, and including a reasonable allowance for sea level rise, does not appear to pose a credible threat of causing an overtopping of the MCR, given the base elevation and height of the structure.

Choice of the best forecasting model of breach discharge (MLM vs. Froehlich methods)

The rate of water flowing through a potential breach, the breach discharge, is predicted in one of two ways: (1) By statistical analysis of historical breach events, leading to a regression equation using reservoir characteristics (typically volume of stored water and pool height) as independent variables; and (2) physics-based hydrodynamic models of the flow of water through a breach of given geometry and rate of growth. The current state-of-practice favors the statistical approach, using the physics-based approach mostly as a check.

Many regression models have been published over the years, but the set of historical data is limited, and most of the models have been calibrated to more or less the same cases. The best-fitting model today appears to be that due to Froehlich[4] which is a bivariate log-log regression of peak discharge against reservoir volume and water depth at the breach.[5] The many competing models either use only one independent variable, or combine volume and depth in a single factor. This leads to lower R^2 statistics and larger predictive errors when compared to Froehlich's bivariate model.[6]

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[4] Froehlich D.C. (1987). "Embankment-Dam Breach Parameters," in *Proceedings of the 1987 National Conference on Hydraulic Engineering*, August 3-7, 1987, R.M. Ragan (Ed.), ASCE, New York. Froehlich D.C. (1995). "Embankment Dam Breach Parameters Revisited," in *Proceedings of the 1st International Conference on Water Resources Engineering*, ASCE, New York.

[5] Wahl, T.L. (2004). "Uncertainty of parameters of embankment dam breach parameters," *Journal of Hydraulic Engineering, 130*(5): 389-397. Wahl, T.L. (1998). "Prediction of embankment dam breach parameters: A literature review and needs assessment, " DSO-98-004, US Bureau of Reclamation, Denver.

[6] A limitation to this conclusion on the superior fit provided by Froehlich's 1995 regression model is that the correlation analysis in the original paper is based on n=22 cases. Of these, only 13 over-

The PNNL analysis uses Froehlich's model. This seems to the reviewer to be the appropriate choice. The NRC non-concurrence uses the McDonald and Langridge-Monopolis (MLM) model because it yields a larger mean estimate of discharge. However, the MLM model provides a poorer fit to historical data than does the Froehlich model and thus seems an inferior choice by a substantial margin (Table 1).

Table 1. Previous Studies of Peak-Outflow Prediction

Investigator	Type	R^2	Number of case studies		Equation and number
			Real	Simulated	
Height of water equations					
Kirkpatrick (1977)	Best-fit	0.790[a]	13	6	$Q_p=1.268(H_w+0.3)^{2.5}$ Eq. 1.1
Soil Conservation Service (SCS) (1981) for dams > 31.4 m	Envelope[b]	Not available	13		$Q_p=16.6(H_w)^{1.85}$ Eq. 2.1
U.S. Bureau of Reclamation (1982)	Envelope	0.724	21		$Q_p=19.1(H_w)^{1.85}$ Eq. 3.1
Singh and Snorrason (1982)	Best-fit	0.488		8	$Q_p=13.4(H_d)^{1.89}$ Eq. 4.1
Storage equations					
Singh and Snorrason (1984)	Best-fit	0.918		8	$Q_p=1.776(S)^{0.47}$ Eq. 5.1
Evans (1986)	Best-fit	0.836	29		$Q_p=0.72(V_w)^{0.53}$ Eq. 6.1
Height of water and storage equations					
Hagen (1982)	Envelope	Not available	6		$Q_p=1.205(V_wH_w)^{0.48}$ Eq. 7.1
MacDonald and Langridge-Monopolis (1984)	Best-fit	0.788	23		$Q_p=1.154(V_wH_w)^{0.412}$ Eq. 8.1
MacDonald and Langridge-Monopolis (1984)	Envelope	0.156	23		$Q_p=3.85(V_wH_w)^{0.411}$ Eq. 9.1
Costa (1985)	Best-fit	0.745[c]	31		$Q_p=0.763(V_wH_w)^{0.42}$ Eq. 10.1
Froehlich (1995)	Best-fit	0.934	22		$Q_p=0.607(V_w^{0.295}H_w^{1.24})$ Eq. 11.1

[a]This R^2 value was calculated using a portion of the writer's original data set.
[b]Wahl (1998) suggests that this is an enveloping equation even though three data points plot slightly above the curve.
[c]This R^2 value was calculated without the five concrete and masonry dams included in the writer's original data set.

Table 1. Statistical fit of various regression models of breach discharge (from Pierce, et al., 2010)

Development of a breach scour hole

The non-concurrence report notes that the development of scour holes at the site of an embankment breach, especially for levees on natural ground, is common. This is true. Furthermore, it seems likely that the formation of a scour hole would increase the breach size and lead to higher outflow discharges. It is not possible to entirely rule out this phenomenon, and the LNNL review probably should have investigated this possibility and its potential effects more thoroughly.

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lap with data used by MLM. The R^2 for these data is 0.93. A re-analysis of the Froehlich regression for the purposes of this review using the n=43 cases published by Wahl (1998) yields an R^2 of 0.72, which is about the same as for the MLM model.

Contravening the possibility of scour hole development are geotechnical site conditions at the embankment. In the geotechnical report (#10) the soils underneath and in the vicinity of the embankment are described as "predominately stiff to hard clays and medium-dense to dense sands." During construction, "cohesive foundation soils having shear strengths less than 1,000 lb/ft^2 were removed and replaced with compacted CL-CH clays," and "any silty or sandy soil encountered within 2 ft below the design subgrade elevation [of the embankment] was excavated and replaced by compacted (CH, CL) clay ... ." Thus, the *in situ* soil conditions are reasonably resistant to large-scale scour, and were further treated during initial site work. While these conditions are no guarantee against the formation of a significant scour hole, they mitigate against scour. The comparative case cited, of the SFWMD Martin Cooling Pond failure does not seem directly relevant as its foundation is uniform sand and the failure mechanism involved piping through the foundation with the formation of sand boils downstream.[7]

Uncertainty in predictions

Even using the Froehlich regression, the uncertainty in predictions peak breach discharge is large, on the order of a ±two-standard-deviation range from ½ to 2. Using the best estimate is not conservative for the purposes of risk assessment. A annual exceedance function of probability is called for, even if the non-concurring review did not identify such.

[7] "Soon after filling, seepage was observed along the downstream slope along nearly 60 percent of the enhancement perimeter. Extensive sand boils developed within the toe ditch and around the pumping station resulting in failure of the embankment. The embankment was washed out over a 600 ft. length allowing the rapid release of 90,000 acre-ft. of water." Bromwell-Carrier Engineers (AMEC). "Martin Cooling Pond," November 1, 2011, http://www.bcieng.com/dams-/-impoundments/martin-cooling-pond.html

Attachment 10a: Response to: Re-Analysis of MCR Breach Flood

June 20, 2011

Response To: Re-Analysis of MCR Breach Flood (June 20, 2011)

Summary

On June 20, 2011, the U.S. Nuclear Regulatory Commission (NRC) provided a document entitled Re-Analysis of the Main Cooling Reservoir (MCR) (hereinafter referred to as the re-analysis document) authored by Dr Hosung Ahn (hereinafter the author). Specifically, the author presents an analysis of the MCR northern-embankment breach (hereinafter referred to as the embankment). The re-analysis document describes the basis for the author's non-concurrence with the analysis described in Section 2.4.4 of the *Safety Evaluation Report (SER)* for South Texas Project (STP) Units 3 and 4 (STP SER); the STP SER analysis was performed by Pacific Northwest National Laboratory (PNNL).

During its review of the re-analysis document, PNNL identified the following significant issues:

1. The author's use of the Manning's n-value for the embankment material is not consistent with PNNL's understanding regarding how the n-value should be selected for National Weather Service (NWS)-BREACH model (hereinafter referred to as the BREACH model) applications.
2. The author's interpretation of the importance of the tail-water cross section (i.e., the cross-sectional shape) on evolution of the breach is not consistent with PNNL's interpretation. PNNL believes that Fread (1991) recommends a tail-water cross section that realistically simulates the backwater effect caused by submergence.
3. The author believes that the postulated MCR breach is comparable with a "river-levee-overtopping breach" case; whereas, PNNL believes that the postulated main cooling reservoir breach is better characterized as a "reservoir-embankment-piping breach" case.
4. The author believes that the MCR breach characteristics should be estimated using McDonald and Langridge-Monopolis's (hereinafter referred to as MLM) approach. PNNL's literature review concluded that the Froehlich method is more appropriate for the MCR.
5. The author believes that 10 CFR 52.79(a)(1)(iii) and 10 CFR Part 50 Appendix A, General Design Criteria 2, call for consideration of "... *the most severe* of the natural phenomena" (*page 3 of the re-analysis document*), while PNNL believes that the phrase highlighted by the author-, "*the most severe*" should be read in the full context of the regulatory language, which is provided below:
    a. 10 CFR 52.79(a)(1)(iii): ... the seismic, meteorological, hydrologic, and geologic characteristics of the proposed site *with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area* and with sufficient margin for the limited accuracy, quantity, and time in which the historical data have been accumulated.... [Note: emphasis in italics added by PNNL]
    b. 10 CFR Part 50 Appendix A General Design Criteria 2, Design bases for Protection Against Natural Phenomena: Structures, systems, and components important to safety shall be designed to withstand the effects of natural phenomena such as earthquakes, tornadoes, hurricanes, floods, tsunami, and seiches without loss of capability to perform their safety functions. The design bases for these structures, systems, and components shall reflect: (1) *Appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area*, with sufficient margin for the

> limited accuracy, quantity, and period of time in which the historical data have been accumulated, (2) appropriate combinations of the effects of normal and accident conditions with the effects of the natural phenomena, and (3) the importance of the safety functions to be performed. [Note: emphasis in italics added by PNNL.]

PNNL's response to the re-analysis document includes this document and a marked-up PDF file entitled Action 13-4 (the re-analysis document). Responses to the four major concerns are included in this document, and minor responses and comments are included in the marked-up PDF file. The first four of these major concerns are discussed in more detail in the following sections.

It should be noted that PNNL's efforts related to preparing the STP SER were, through confirmatory analyses, to make an informed determination regarding the applicant's established design-basis flood water surface elevation. Further, PNNL was asked to determine whether the approach was reasonable, conservative, and site-specific.

To estimate peak water surface elevations within the area of concern at STP Units 3 and 4, the location and width of the embankment breach and the discharge hydrograph through the embankment were estimated by the applicant, South Texas Project Nuclear Operating Company (STPNOC). The applicant, PNNL, and the author have performed this step using a variety of methods described in the published literature. These methods produce estimates of the timing of the breach; the peak discharge; the discharge hydrograph; the average breach width at peak discharge and the final, fully-developed breach width; top and bottom breach widths at peak discharge, and final width, which are described by Wahl (1994). This analysis is an intermediate step to establishing a reasonable and conservative estimate of the peak water surface elevation at the STP Unit 3 and 4 site where safety-related facilities are to be located; this latter step in the analysis is separate from the discussion in this section.

PNNL's process for determining the reasonableness of the applicant's approach as described in the Final Safety Analysis Report (FSAR), in responses to requests for additional information, and from the applicant's presentations at site audits is included in the SER. This process involved acquiring and reviewing relevant material provided by the applicant, obtained from the published literature, and resulting from application of the BREACH model. PNNL's process for determining a specific analytical approach was developed out of this review to establishi whether the applicant's approach was reasonable, conservative, and site-specific as opposed to whether the applicant had applied every potential method using a compounded set of the most conservative parameters in combination.

1: Embankment Material Characterization in NWS BREACH

The applicant, PNNL, and the author used several methods, including the BREACH model, for estimating the embankment breach geometry. The roughness parameter (also referred to as Manning's n-value, or just the n-value) is used two different ways in the BREACH model (Fread 1991a). The first way corresponds to the more traditional flow-routing approach in which a segment of river or flood plain is analyzed. In the BREACH documentation, this is referred to as the roughness parameter for the tail-water cross section. In the second way, the BREACH model uses the roughness parameter to estimate the erosion of the embankment. In the BREACH documentation, this is referred to as the roughness parameter for the embankment material. These roughness parameters do not have to be equal nor do they represent the same physical process.

Sensitivity analyses performed by PNNL and reported in the STP SER, establish that larger n-values for the embankment produce larger peak embankment discharges. In the sense that larger peak discharge are likely to result in higher peak water surface elevations in the area of interest, the use of larger embankment n-values yields more conservative estimates.

Meader, who is a consultant at BOSS International, which is a distributor of BREACH model,[1] states:

> *BREACH requires that you define the Manning's roughness value for the cross section downstream of the reservoir, and the Manning's roughness value of the earthen material that makes up the dam. The cross section roughness simply represents the on-site field conditions that the flow will encounter after it leaves the failed reservoir structure. This should be the same as what you enter into DAMBRK.* ***The dam material roughness represents the roughness of bare earth, which can be quite small since earthen material (clay, etc.) tends to be quite smooth.***

Several applications of BREACH seem to conservatively follow Meader's guidance. Singh (1996) described the use of BREACH for failure analyses of dams caused by natural catastrophic events: (1) the breach from failure of a natural dam resulting from a landslide on the Montaro River in Peru and (2) a breach at Spirit Lake in Washington State resulting from the eruption of Mt. Saint Helens in 1984. Singh reported n-values of 0.02 and 0.018 for his analyses of the Montaro and Spirit Lake events, respectively. In a study using the DAMBRK model, which was a precursor to the BREACH model, Narasimham et al. (2011) used an n-value of 0.05 to analyze the Alamatti Dam failure in India. Although Ramesh and Rao used an n-value of 0.05 in their work, it is useful to point out that DAMBRK uses empirical equations to simulate growth of the breach in an earthen embankment. These growth equations do not use the Manning's roughness factor as a parameter. The n-value used in DAMBRK is specifically intended to represent the characteristics of the channel and overbank areas downstream of the dam (Fread 1991b).

References to the literature (Chow 1955; Arcement and Schneider 1990) to guide reasonable estimation of the base embankment material roughness (as used in the BREACH model) should be limited to n-values for bare earth. This approach specifically excludes considerations of existing vegetation, channel meanders, exiting obstructions, etc. In contrast, the approach advocated by the author is based on the inclusion of these additional conditions. During its confirmatory analysis, PNNL increased the base value of the embankment material parameter value to obtain a very conservative value. Arcement and Scheider (1990) suggest n-values ranging from 0.012 (for flow over fine sand or concrete) to 0.07 (for flow over boulders) (see Table 1 in Arcement and Scheider [1990]).

Based on these examples and guidance from a consultant, PNNL believes that the n-value used by the applicant (i.e., 0.05) is reasonable and conservative, and for more realistic simulations, the value could be reasonably set at an even lower values as are used in the reference case studies. PNNL acknowledges that even larger values of the embankment roughness parameter would likely produce larger peak discharges; however, we also believe that the use of larger values would be implausible. In addition, increasing the n-value for the embankment material to attain an appropriate level of conservatism, based on inclusion of flow features not associated with embankment material, is not appropriate and is counter to guidance provided to users of the BREACH model.

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[1] Source: http://www.bossintl.com/forums/31462-post2.html.

PNNL believes that the applicant's n-value selection for the embankment material is reasonable, conservative, and consistent with historical applications of the BREACH model, and is consistent with the guidance of experts.

2: Importance of Downstream Control in NWS BREACH

The author believes the tail-water cross section should be specified as the valley cross section downstream of the dam. Fread (1991) recommends a correction to account for the downstream submergence effect. This effect would result in a higher water surface elevation at the breach section and a smaller discharge. The tail-water n-value is used in the BREACH model to compute the tail-water depth using Manning's equation. Note that the tail-water n-value used here is different than the value used to characterize the embankment material. PNNL's review of Fread (1991) concluded that specifying a realistic tail-water cross section is necessary to simulate a breach discharge hydrograph. The tail-water cross section, particularly for a piping failure, should be specified on the downstream face of the embankment and not taken as the whole width of the valley downstream of the embankment. PNNL believes that the tail-water cross section should be specified as recommended by Fread (1991).

3: Levee and Embankment Failures and Comparison with Historical Embankment Breaches

The author uses the term "levee" throughout the re-analysis document to refer to the MCR embankment. PNNL does not consider the MCR embankment to be a typical levee and avoids its use in the MCR breach analysis. The applicant also uses the term "embankment," not "levee" in the FSAR. The U.S. Army Corps of Engineers (2000) states:

> *The term levee as used herein is defined as an embankment whose primary purpose is to furnish flood protection from seasonal high water and which is therefore subject to water loading for periods of only a few days or weeks a year. Embankments that are subject to water loading for prolonged periods (longer than normal flood protection requirements) or permanently should be designed in accordance with earth dam criteria rather than the levee criteria given herein.*
>
> *Even though levees are similar to small earth dams they differ from earth dams in the following important respects: (a) a levee embankment may become saturated for only a short period of time beyond the limit of capillary saturation, (b) levee alignment is dictated primarily by flood protection requirements, which often results in construction on poor foundations, and (c) borrow is generally obtained from shallow pits or from channels excavated adjacent to the levee, which produce fill material that is often heterogeneous and far from ideal. Selection of the levee section is often based on the properties of the poorest material that must be used.*

Based on his review of historical levee failure cases (examples provided by the author include levee failures along Eastern European Rivers and a delta levee system in California), the author made the following statement in the re-analysis document:

From these levee breach records, we can conclude that levee breach widths are generally wider than those of dams and that potential levee breach widths could be wider than 1000 ft. This conclusion supports the use of the MLM breach width equation for the MCR breach scenario.

PNNL believes it is not surprising that levees experience wider breaches when compared with embankment dams used for static storage. Levees generally are used to contain discharges within stream channels and fail by overtopping when flood levels exceed the design bases. When large floods cause levee failures, the flood overtops the levee along significantly long distances over the length of the levee. The duration of overtopping also can be significantly long, depending on the magnitude of the flood compared to the levee's design basis. A breached levee also experiences substantial shear forces from the flood discharge as the river flows along its length and head cutting from floodwaters that overtop the levee. These conditions promote a longer levee breach. In the re-analysis document, the author seems to conclude that the MCR embankment would fail in a similar way and that accompanying conditions would be similar to a river levee overtopping failure. PNNL disagrees with that conclusion. The MCR is a static storage pond that does not experience large floods with substantial upstream areas contributing runoff. It has been shown that the MCR would not overtop (see Sections 2.4S.8 and 2.4.8 of the STP FSAR and NRC SER, respectively). As the MCR would drain after a postulated embankment failure (due to piping), the stored water would discharge through the breached section, and virtually no water would discharge along the embankment. Because the MCR does not fail by overtopping, erosion of the embankment would proceed laterally from the single eroded channel (i.e., the pipe) to expand the breach; long lengths of the embankment would not be eroded as would happen in an overtopping event.

PNNL believes that the author's comparison of the MCR embankment piping failure to levee breaches is not valid. Therefore, PNNL re-examined the other embankment and dam breaches that the author mentioned in the re-analysis document to evaluate how they compare with the postulated MCR event.

PNNL objectively reviewed the DSO database for historical dam breaches. In this database multiple entries were provided for the same event when different reports presented different values for the same breach characteristics. In cases for which multiple entries existed, the database also identified the entry with the most reliable information. As described above, PNNL conducted an objective search of the DSO database to develop a reasonable comparison with the postulated MCR embankment breach.

The MCR volume and water depth prior to the breach were used as variables to search for similar records in the DSO database. Selection of the variables (the volume of water released through the breach and the difference between the initial reservoir water elevation and the elevation at the base of the breach) used for this comparative analysis was based on a review of the literature (Wahl 1998; Froehlich 1995; Froehlich 1987; MacDonald and Langridge-Monopolis 1984) as these variables are the most reasonable predictors of breach characteristics. The choices of variables were objective and were supported by information in the published literature.

One method typically used to characterize breach parameters is to review historical databases. These databases include historical events that differ in terms of structure (dams, embankments, levees) and impoundments (artificial reservoirs, impounded streams), and causal mechanisms (geotechnical, construction types, hydrological events, and cascading failures). Wahl (1998) identified comparative analysis as an approach that had been used. Comparative analysis, along with parametric models and predictor equations, rely on a small number of breach cases. Wahl (1998) contrasts this approach with a physically based approach (e.g. the BREACH model) and notes that comparative analyses rely on databases of dam failure case studies and that these databases are small and contain few cases of very high dams or very large storage volumes. PNNL's use of a comparative analysis approach is not intended to replace a physically-based approach for estimating breach parameters; rather, it is provided to determine if the physically based model analysis produces a plausible set of breach parameters.

The State of Colorado (2010) described a comparative analysis approach used to estimate dam breach parameters. This approach was objectively followed by PNNL. The bases of the PNNL search of the Reclamation Dam Safety Office (DSO) database are described in the STP SER and are repeated here for convenience. For the MCR analysis, the height of water above the breach is 21.9 ft, and the stored volume above the breach invert is 152,700 ac-ft. To reasonably bound the corresponding characteristics of the MCR, PNNL searched the DSO database to select entries with a height of water above the breach between 15 to 50 ft and a stored volume above the breach bottom between 100,000 to 300,000 ac-ft. Our search of the DSO database identified the Martin Cooling Pond breach in Florida. Based on a comparison of the Martin Cooling Pond failure with the postulated failure of the MCR, PNNL concluded that the MCR breach parameters predicted using the BREACH model are reasonable.

PNNL's comparative analysis identified the Martin Cooling Pond breach as the most comparable historical breach to the postulated MCR northern embankment breach. The author identified several other historical breaches in the re-analysis document. Each of these breaches is described below.

Martin Cooling Pond, Florida

AMEC-BCI provides the following description of the Martin Cooling Pond[1]:

> *The Martin Cooling Pond is a 6,900 acre reservoir located east of Lake Okeechobee and north of the St. Lucie Canal in Martin County. The reservoir has an embankment length of 17.5 miles with a maximum design head of 23 feet.*
>
> *Soon after filling, seepage was observed along the downstream slope along nearly 60 percent of the enhancement perimeter. Extensive sand boils developed within the toe ditch and around the pumping station resulting in failure of the embankment. The embankment was washed out over a 600 ft length allowing the rapid release of 90,000 acre-ft of water.*

Wahl (1998) noted that the Martin Cooling Pond average breach width was 186 m (610 ft), and the peak flow was 3115 m^3/s (110,005 ft^3/s); he also reported that the cause of the breach was a foundation defect. These data are reasonably consistent with those presented by the South Florida Water Management District (SFWMD) (SFWMD 1980). The SFWMD also describe the scour hole created by the breach discharge.

The Martin Cooling Pond breach was attributed to a flaw in the design of the embankment. The pond is similar to the MCR in that it is a water storage pond rather than an impounded river or creek. A description of possible causes of the breach and an account of the resultant flooding were provided in the *Palm Beach Post* on May 15, 2006.[2] The average breach width and peak discharge are similar to those estimated by PNNL for the postulated failure of the MCR embankment.

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[1] Source: http://www.bcieng.com/dams-/-impoundments/martin-cooling-pond.html.

[2] Source: http://www.palmbeachpost.com/treasurecoast/content/local_news/epaper/2006/05/15/s1a_FPL_DIKE_0515.html.

Swift Reservoir, Birch Creek, Montana

In their report, Boner and Stermtz (1967) noted:

> *The floods in the valley of Birch Creek were extremely severe because of the failure of the Swift Dam and the discharges of the contents of Swift Reservoir into the valley. The maximum capacity of the reservoir was approximately 30,000 acre-ft, and evidence suggests that the dam failed suddenly. The peak discharge of 881,000 cfs from an area of 105 square miles was determined by indirect measurements near Dupuyer, Montana, which is 17 miles downstream. The force of the flood was remarkably demonstrated on the valley floor of Birch Creek in the half-mile reach directly below the dam. Several large blocks of rockfill material from the dam were moved a quarter to half a mile along the valley floor without being broken up. The stratification in the fill material was still visible in these large blocks in September 1964. The largest block observed contained approximately 475 cubic yards of material and several others exceeded 100 cubic yards. The angular rock fragments in the fill material from the dam contrast sharply with the rounded cobbles and gravel of the flood plain on which they came to rest. Farther downstream on Birch Creek north of Valier, the channel was widened approximately 70 feet near the bridge crossing, and scour of the channel bed into bedrock probably exceeded 5 feet.*

The failure of the Swift Reservoir differs from that postulated for the MCR. Unlike the postulated MCR breach, the Swift Reservoir failure was caused by overtopping conditions, and the dam impounded a stream.

Machhu II Dam/Machhu River, India

Although the DSO database lists seepage as the best reliable information, local reports from other literature sources clearly state that the Machhu II Dam failed because of overtopping (Dhar et al, 1981)

Professor B.S. Thandaveswara, Indian Institute of Technology Madras, noted that:

> *This dam was built near Rajkot in Gujarat, India, on River Machhu in August, 1972, as a composite structure. It consisted of a masonry spillway in river section and earthen embankments on both sides. The embankment had a 6.1 m top width, with slopes 1 V :3 H and 1 V : 2 H, respectively, for the upstream and downstream slopes and a clay core extending through alluvium to the rocks below. The upstream face had a 61 cm small gravel and a 61 cm hand packed riprap. The dam was meant to serve an irrigation scheme. Its storage capacity was 1.1×10^8 m^3. The dam had a height of 22.56 m above the river bed, a 164.5 m of crest length of overflow section, and a total of 3742 m of crest length for the earth dam.*

Dhar et al. (1981) described the rainfall event that caused the overtopping failure of the Machhu II Dam embankment. This dam impounded the Machhu River and was designed to pass flows of 57,000 m^3/s (2,012,936 ft^3/s). When excessive rainfall occurred in the watershed above the dam, a peak flow of 140,000 m^3/s (4,944,053 ft^3/s) overtopped the hand-packed embankments on the flanks of the dam. The resultant discharge through the breach continued for at least three days (Dhar et al. 1981). Dhar et al. (1981) also concluded that the breach was caused by antecedent basin conditions of moderate to heavy rainfall.

Wahl et al. (2008) states:

At this time, the database includes 13 case studies of embankment failure by overtopping erosion, with plans to include internal erosion failures in the future. A range of materials is represented, from well compacted embankments with a cohesive core (Oros Dam) to ***non-compacted hand-made embankments (Machhu 2 Dam)****.*

The Machhu II Dam failure case differs from postulated the MCR breach in terms of the construction of the embankment. The Macchu II Dam failure was caused by overtopping. In addition, the Macchu II Dam impounded a stream that was subject to flooding in contrast to the MCR, which is not subject to flooding.

Salles Oliveira, Brazil

The DSO database lists overtopping as the failure mechanism for the Salles Oliveira Dam. Franca and Almeida (2005) provide a discussion of a paper by Rozov (2003) in which a final breach width of 131 m (430 ft) and a peak discharge breach width of 37 m (121 ft) are reported. Gilbert and Miller (1991) summarize the Salles Oliveira breach as follows:

Located 10 km downstream of Euclides de Cumba Dam, Armando de Salles Oliveira Dam is 660 m long with two 35-m-high homogeneous earth fills of rolled clay with inclined and horizontal drains.

Overtopping data: Date – 20 January 1977; maximum depth – 1.3 m; duration – 15 to 30 mins.

Damage: When the Euclides de Cumba Dam began breaching, a flood wave formed which overtopped and failed the Armando de Salles Oliveira Dam 15 to 30 min later. Breaching occurred at the right abutment of the right embankment and through a low saddle embankment on the reservoir rim. Outside of the breached area, there was very little damage to the downstream slope. Approximately one-third of the right embankment was removed.

Gilbert and Miller also published a table showing the ratio of breach width vs. crest length (Gilbert and Miller 1991;Table 5, page number 44 and 45, pdf pages 48 and 49). For the Salles Oliveira case, they provide the following characteristics:

- Dam height: 115 ft
- Reservoir capacity: 21,000 ac-ft
- Peak discharge: 254,000 cfs
- Breach width: 550 ft
- Breach depth: 115 ft
- Time of failure: 2.0 hr
- Cause of breach: severe rainstorms
- Crest width: 2165 ft.

Unlike the MCR, the breached dam was located on a river subject to rainfall-induced flood events. The water depth was higher than the depth of the MCR.
Table 1 presents a comparison of reservoir parameters of selected historical breach cases.

Table 1. Comparison of Reservoir Parameters of Selected Historical Breach Cases

Case	Reservoir Volume (m^3) [acre-feet]	Initial Water Depth (m) [ft]	Final Breach Width (m) [ft]	Peak Discharge (m^3/s)[cfs]	Failure Type
Main Cooling Reservoir	1.88×10^8 [1.53×10^5]	6.7 [21.9]	127 [417]	3,681 [130,000]	Piping[1]
Martin Cooling Pond	1.4×10^8 [1.13×10^5]	8.5 (28)	186 [610]	3,115 [110,005]	Piping/ Foundation Defect
Salles Oliverira	7.15×10^7 [5.80×10^4]	38.4 [126]	168 [551]	7,200 [254,266]	Overtopping
Swift Reservoir	3.70×10^7 [3.0×10^4]	47.9 [160]	225 [738]	24,947 [881,000]	Overtopping
Machhu 2	1.1×10^8 [8.92×10^4]	22.6 [74]	Not reported	26,650 [941,136]	Overtopping/ Seepage
[1] MCR failure is postulated.					

Breach Width Conservatism

The State of Colorado (2010) used a comparative-analysis approach to estimate dam breach parameters. This comparative-analysis approach was objectively followed by PNNL to identify similarities between the Martin Cooling Pond case and the MCR case, and then to verify that the estimated breach width for the MCR as estimated by the BREACH model was a reasonable value.

In its Guidelines for Dam Breach Analysis, the State of Colorado (2010) states:

> *Discussion in the Washington State Technical Note suggests a minimum breach formation factor (BFF) of 100 ac-ft^2. This method [the Washington State method] therefore appears more suited to Small or Large dams, while the MacDonald and Langridge-Monopolis method appears to be more appropriate for Minor dams and some Small dams with a BFF less than 100 ac-ft^2.*

The author used a reservoir head of 21.9 ft and storage volume of 6.653×10^9 ft^3; the product of these two numbers yields the MCR breach formation factor of 3.34×10^6 acre-ft^2.

Scour Hole

The author points out that development of the scour hole could cause the embankment to fail more extensively than it otherwise would. In its comparative analysis, PNNL used the DSO historical dam-breach database that included earth-fill embankment breaches during which development of a scour hole is more likely to occur. The database implicitly includes the effects of scour-hole development on breach geometry. Models based on regression analysis are derived from fits to these historical cases. Results from process models (e.g., the BREACH model) compare reasonably well with findings from comparative analysis and accepted regression models. Therefore, PNNL determined that, in terms of breach geometry and peak flows, the effects of a scour hole were reasonably addressed by the applicant.

The SFWMD (1980) reported the dimensions of the scour hole resulting from the Martin Cooling Pond embankment failure:

> *As the flood wave escaped through the breach, it removed about 100,000 cubic yards of the naturally occurring fine grained sand from the beneath and west of the breach. The resulting scour hole was roughly rectangular in plan, about 450 feet wide (parallel to the dike) by about 700 feet long (perpendicular to the dike). The deepest part of the scour hole was located in the center and west of the breach. This deepest hole was also rectangular in plan, 190 feet wide by 400 feet long. The invert of this hole was at elevation -7 feet at the east end; it sloped upward to an elevation of about +4 feet at the west end.*

Boner and Stermitz (1967) reported the scour that resulted from the Swift Reservoir breach. They stated that downstream of the dam "... scour of the channel bed into bedrock probably exceeded five feet."

4: Review of Empirical Approaches for Dam-Breach Characterization and Associated Prediction Uncertainty

In their publication, MLM (1984) described a method for estimating breach parameters. In another publication, Wahl (2004) described many approaches for estimating breach parameters including the regression approaches of Froehlich, MLM, and Bureau of Reclamation, and physically based approaches including the BREACH model (Fread 1991a).

In his article, Wahl (2004) reviews methods for predicting breach parameters. These methods are based on regression analyses (see Wahl 1998), are physically based (i.e. the BREACH model), or are simplified methods for reconnaissance-level methods (e.g. MLM). Gee (2010) reviews these and other methods and discusses the need for including a breach process-based model into flood-routing models (e.g., the U.S. Army Corps of Engineers Hydrologic Engineering Center River Analysis System [HEC-RAS]). Table 2 provides a comparison of uncertainty estimates for breach flow parameters and peak flow equations based on Wahl's review (Wahl 2004).

Table 2. Uncertainty Estimates for Breach Flow Parameters and Peak Flow Equations

Approach	Parameter	Prediction Interval Around a Hypothetical Predicted Value of 1.0
MLM (1984) (earthen)	Volume Eroded	0.15 - 6.8
Froehlich (1987, 1995)	Average Width	0.40 - 2.4
MLM (1984)	Time of Failure	0.24 - 11.0
Froehlich (1987, 1995)	Time of Failure	0.38 - 7.3
MLM (1984)	Peak Discharge	0.15 - 3.7
Froehlich (1987, 1995)	Peak Discharge	0.53 - 2.3

Table 2 shows the spread of observed breach parameters around MLM (1984) and Froehlich (1987, 1995) regression fits for the data in terms of the volume of embankment eroded (MLM only), breach width (Froehlich only), time of failure, and peak discharge. While it is possible with knowledge of the embankment geometry to compute average breach width from MLM's prediction of the eroded embankment volume, Wahl (2004) notes that:

> *Using the MacDonald and Langridge-Monopolis equation, the estimate of eroded embankment volume and associated breach width for the top-of-joint-use scenario is also comparable to the other equations. However, for the top-of-flood-space scenario, the prediction is much larger than any of the other equations, and in fact is unreasonable because it exceeds the dimensions of the dam.*

After comparing four methods (i.e. Froehlich [1987, 1995], Von Thun and Gillette [1990], Reclamation [1988], and MLM [1984]) for estimation of breach parameters, Wahl (2004) states:

> *In fact, the Froehlich equation has both the lowest prediction error and smallest uncertainty of all the peak flow prediction equations.*

Gee (2010) compared six methods for estimating breach width, time of failure, and side slope for three breach cases. Three of the methods were still being tested when Gee's paper was presented, and are considered to be preliminary. For the Oros Dam breach case in Brazil, the reported breach width was 200 m. Estimates of the breach width determined using the MLM (1984) and Froehlich (1987, 1995) approaches and the BREACH model were 900, 305 and 284 m, respectively. For the Banqio breach case in China, the reported breach width was 210 m and the corresponding MLM, Froehlich, and BREACH model estimates were 1037, 281, and 641 m, respectively. In a small laboratory setting (Norway4 case), a 10-m breach was created; for this case, the MLM, Froehlich and BREACH estimates were 3.5, 2.8, and 2.1 m, respectively. For these historically-reported breaches, all of the established methods over-predicted the breach width; therefore, in these cases, results obtained from use of these methods can be considered to be conservative in the sense that breaches that are wider likely may yield larger peak discharges.

Pierce et al. (2010) evaluated regression relationships used to estimate dam breach geometric and temporal parameters. They concluded that:

> *A comparison of selected historical and newly-derived expressions indicates that the Evans (1986), Reclamation (1982), and Froehlich (1995) relations remain valid for conservative peak-outflow predictions.*

PNNL still believes that the applicant's use of the Froehlich regression fit to estimate MCR breach parameters is reasonable and conservative, and is consistent with expert opinion found in the literature.

References

Arcement G.L. and V.R. Schneider. 1990. *Guide for Selecting Manning's Roughness Coefficient for Natural Channels and Flood Plains*. United States Geological Survey Water-Supply Paper 2339. Available at www.fhwa.dot.gov/bridge/wsp2339.pdf.

Boner F.C. and F.Stermitz. 1967. Floods of 1964 in the United States. U.S. Geological Survey Water-Supply Paper 1840-B. Prepared in cooperation with the State of Montana and agencies of the Federal Government.

Chow V. T. 1959. *Open Channel Hydraulics.* McGraw-Hill, New York.

Dhar O.N., P.R. Rakhecka, B.N. Mandal, and R.B. Sangam. 1981. "The Rainstorm Which Caused the Moriv Dam Disaster in August 1979." *Hydrological Sciences Bulletin* 26(1):72-81.

Franca M.J. and A.B. Almeida. 2005. "Discussion of Modeling of Washout of dams (by A.I. Rozov, 2003)." *Journal of Hydraulic Research* 43(4):439-444.

Fread D.L. 1991a. *BREACH: An Erosion Model for Earthen Dam Failures.* National Weather Service, National Oceanic and Atmosphereic Administration, Silver Springs, Maryland.

Fread D.L. 1991b. *The NWS DAMBRK Model: Theoretical Background/User Documentation.* Revision 4, National Weather Service, National Oceanic and Atmospheric Administration, Silver Springs, Maryland.

Froehlich D.C. 1987. "Embankment-Dam Breach Parameters. Hydraulic Engineering 1987." In *Proceedings of the 1987 National Conference on Hydraulic Engineering*, August 3-7, 1987, R.M. Ragan ed., American Society of Cival Engineering, New York.

Froehlich D.C. 1995. "Embankment Dam Breach Parameters Revisited." In *Proceedings of the 1st International Conference on Water Resources Engineering*, American Society of Civil Engineers, New York.

Gee D.M. 2010. "Use of Breach Process Models to Estimate HEC-RAS Dam Breach Parameters." Presented at 2nd Joint Federal Interagency Conference, June 27-July 1, 2010, Las Vegas, Nevada.

Gilbert P.A. and S.P. Miller. 1991. *A Study of Embankment Performance During Overtopping.* Technical Report GL-91-23, U.S. Department of the Army Waterways Experiment Station, U.S. Corps of Engineers, Vicksburg, Mississippi.

Harza Engineering Company. 1984. *Evaluation of Strength Parameters and Stability: Main Cooling Reservoir Embankment, Volume 1 Report.* Prepared for Bechtel Energy Corporation South Texas Project.

MacDonald T.C. and J. Langridge-Monopolis. 1984. "Breaching Characteristics of Dam Failures." *Journal of Hydraulic Engineering* 110(5):567-586.

Narasimham, M.L., T.S. Rao and M.V.V. Rao. 2011. Dam Break Analysis of Alamatti Dam. Available at http://tshivajirao.blogspot.com/2011/01/almathi-dam-hazardous.html.

Next Big Future. March 23, 2011. Hydroelectric Dam Failures-Fujinuma Dam in the Recent Japan Earthquake. Available at http://nextbigfuture.com/2011/03/hydroelectric-dam-failures-fujinuma-dam.html.

Philip Williams and Associates. 2008. *Silver Creek Dam Break Analysis: Final Report. Prepared for the City of Silverton, Oregon.* Portland, Oregon.

Pierce M.W., C.I. Thorton, and S.R. Abt. 2010. *Predicting Peak Outflow from Breached Embankment Dams*. Prepared for National Dam Safety Review Steering Committee on Dam Breach Equations. Colorado State University. Fort Collins, Colorado.

Singh V.P. 1996. *Dam Breach Modeling Technology*. Kluwer Academic Publishers, The Netherlands. ISBN 0-7923-3925-8. http://books.google.com/books?id=pPXj0ka9bZYC&pg=PA169&lpg=PA169&dq=nws-breach+roughness&source=bl&ots=eCw5ka7scT&sig=FlVSqUdZ4-8mWCGIcoWmrmjr4is&hl=en&ei=W9UcTq3AG4TCsAPRv4CjDA&sa=X&oi=book_result&ct=result&resnum=7&ved=0CEEQ6AEwBjgK#v=onepage&q=Mannings&f=false.

South Florida Water Management District (SFWMD). 1980. *Interim Final Draft Report on Embankment Failure*. Florida Power and Light Company Martin Plant Cooling Pond.

State of Colorado. 2010. *Guidelines for Dam Breach Analysis*. Department of Natural Resources, Division of Water Resources, Office of the State Engineer, Dam Safety Branch. Denver, Colorado.

State of Montana. 2007. Update to the State of Montana Multi-Hazard Mitigation Plan and Statewide Hazard Assessment. Available at http://dma.mt.gov/des/Library/2007_PDM_Plan_Update/State_PDM_Plan.htm.

Thandaveswara B.S. Undated. Indian Institute of Technology Madras-Hydraulics. Available at http://nptel.iitm.ac.in/courses/IIT-MADRAS/Hydraulics/pdfs/Unit41/41_2.pdf.

U. S. Army Corps of Engineers. 2000. *Engineering and Design: Design and Construction of Levees*. EM 1110-2-1913. Washington, D.C.

U.S. Bureau of Reclamation 1982. "Guidelines for defining inundated areas downstream from Bureau of Reclamation dams." Reclamation Planning Instruction No. 82-11, U.S. Department of the Interior, Bureau of Reclamation, Denver, Colorado, 25p.

U.S. Bureau of Reclamation 1988. Downstream hazard classification guidelines. ACER Technical Memorandum No. 11, U.S. Department of the Interior, Bureau of Reclamation, Denver, Colorado, 57p.

Von Thun, J. L., and Gillette, D. R. 1990. "Guidance on breach parameters." Internal Memorandum, U.S. Dept. of the Interior, Bureau of Reclamation, Denver, 17p.

Wahl T.L. 2004. "Uncertainty of Predictions of Embankment Dam Breach Parameters." *Journal of Hydraulic Engineering* 130(5)389-397.

Wahl T.L. 1998. *Prediction of Embankment Dam Breach Parameters: A Literature Review and Needs Assessment*. DSO-98-004, U. S. Department of the Interior, Bureau of Reclamation Dam Safety Office, Denver, Colorado.

Wahl T.L., J-R Courivaud, , R. Kahawita, G.J. Hanson, M.W. Morris, and J.T. McClenathan. 2008. "Development of Next-Generation Embankment Dam Breach Models." In *The Sustainability of Experience – Investing in the Human Factor*, Proceedings of 28th Annual USSD Conference, April 28-May 2, 2008, Portland, Oregon, U.S. Society on Dams. Denver, Colorado.

Attchment 10b: Summary of Comments on Action 13-4 (PNNL markup of June 20th submittal by H. Ahn, item 11c)

Enclosure 1: Re-Analysis of MCR Breach Flood

(Attachment to the Non-concurrence on the Safety Evaluation Report Section 2.4.4 for the South Texas Project Units 3&4 Combined License Application)

June 20, 2011

Hosung Ahn, Ph.D., P.E.

U.S. Nuclear Regulatory Commission

NRO/DSER/RHEB

Summary of Comments on Action13-4_PNNLMarkUp (3).pdf

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Table of Contents

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1 Executive Summary

This document addresses one of the three non-concurring issues raised by Dr. Hosung Ahn on the Safety Evaluation Report (SER) Chapter 2.4 for the South Texas Project (STP) Combined License Application (COLA). This report address the issue related to determining a site parameter - design basis flood level - which is critical for structural design and flood protection of safety-related facilities.

STP provided in their Final Safety Analysis Report (FSAR) the estimation of the design basis flood (DBF) level caused by a postulated main cooling reservoir (MCR) breach. NRC staff reviewed the estimation of the DBF elevation and concluded in the SER that the applicant's DBF estimation is acceptable. PNNL ~~help~~ the NRC staff in reviewing the STP's MCR breach analysis. PNNL performed a breach modeling as a confirmatory analysis~~,~~ however their breach modeling includes a serious error. It is my opinion that the postulated main cooling reservoir breach scenario and design basis flood level estimation in both FSAR and SER are neither accurate nor conservative. To justify my technical position and to provide a potential solution, I performed a re-analysis of the MCR breach flood.

The applicable regulations in estimating the design basis flood level are: (a) 10 CFR 52.79(a)(1)(iii) and GDC 2 of Part 50, as they relate to identifying hydrologic site characteristics with appropriate consideration of **the most severe** of the natural phenomena; and (b) 10 CFR 100.20(c)(3), as it requires to identifying and characterizing the **maximum probable** hydrologic events such as wind speed and precipitation for site safety analysis.

This report includes all available relevant information on the MCR breach analysis and modeling. The analysis done in this report is very comprehensive – It is a multiple-step process including the following modeling:

- Levee breach parameter estimation using all available empirical equations
- Validation of breach parameters and outflows using the BREACH model
- Simulating breach outflow using the FLDWAV model, and
- Routing of breach flood at the site using the FLO-2D model.

The result of the breach and flood modeling indicates that the reasonably conservative DBF levels at the proposed Units 3 & 4 would be about 45 ft MSL considering ~~a~~ enough margin for the wind wave setup and modeling uncertainty. This new estimate is much higher than the STP's DBF estimation of 40 ft MSL. As this increase in flood level is significant in structural design and flood protection measures, the author of this report strongly recommend revising both the FSAR and the current version of the SER.

In addition to site specific data, this report includes a lot of useful information in relation to the ~~levee~~ breach flood analysis in general. However, the reader~~ships~~ of this report should note that the content of this report could be applied to other levee breach flood analyses but not directly used because the levee breach flooding is governed by many site-specific parameters and conditions.

Page: 3

Number: 1 Author: D3B102 Subject: Comment on Text Date: 7/11/2011 2:59:36 PM -04'00'
NRC suggested not using MCR as it may lead to confusion with "Main Control Room"

Number: 2 Author: D3B102 Subject: Replacement Text Date: 7/11/2011 2:57:23 PM -04'00'
helped

Number: 3 Author: D3B102 Subject: Replacement Text Date: 7/11/2011 3:02:30 PM -04'00'
change "," to ";"

Number: 4 Author: D3M760 Subject: Highlight Date: 7/19/2011 8:07:47 PM -04'00'
The regulation includes consideration not of the MOST SEVERE but the MOST SEVERE THAT HAVE BEEN HISTORICALLY REPORTED FOR THE SITE. It also includes consideration of margin of error.

Number: 5 Author: D3M760 Subject: Highlight Date: 7/19/2011 8:07:47 PM -04'00'

Number: 6 Author: D3M760 Subject: Inserted Text Date: 7/19/2011 8:07:47 PM -04'00'

Number: 7 Author: D3B102 Subject: Cross-Out Date: 7/11/2011 3:29:08 PM -04'00'

Number: 8 Author: D3B102 Subject: Cross-Out Date: 7/11/2011 3:29:27 PM -04'00'

2 Introduction

Final Safety Analysis Report (FSAR) Section 2.4.4 of the South Texas Project (STP) combined licensing application (COLA) addresses potential dam failures. The purpose of the dam failure analysis in FSAR is to ensure that potential external flood hazard to safety-related structures due to failure of dam is considered in the plant design and flood protection measures. This FSAR section covers postulation of dam failure scenarios, hydrodynamic analysis of dam failures, and determination of flood characteristics (e.g., flood level, duration, velocity, etc.) at the site. The dam failure flood analysis used in FSAR 2.4.4 is a deterministic approach – We assume a dam failure 1 without considering the condition of the dam or probabilities of failure occurrence and consequence.

The dam failure analysis in the STP's FSAR includes the potential main cooling reservoir (MCR) embankment breach and the effects of the breach on the site flooding. STP identified that the MCR breach is the controlling design basis flood (DBF) scenario among 2 all potential flood events, including rainfall, hurricane, tsunami, and dam breaches. 3 They also identified that the controlling flood level is higher than the bounding design flood level of one feet below the plant grade specified in the ABWR Design Certificate Document (DCD), thus handled the DBF level as a DCD departure. The ABWR DCD requires that the maximum flood level should be 1 ft below the plant grade.

The MCR covers an area of approximately 3 miles by 4 square miles and is formed by an embankment levee about 12.4 miles long. The area of the MCR is about 7,000 acres. For the Units 3&4, STP proposed to use the MCR which was originally built for four reactor units. The MCR was designed for the normal maximum operation level of 49 ft mean sea level (MSL). However, STP has been operating the reservoir at a pool level of 45 ft MSL after a field seepage test in early 1990s. With the proposed two units, STP proposed to raise the operating reservoir level from 45 ft MSL to 49 ft MSL. 4 ~~The raise~~ 6 of 5 the operating level could increase the potential ~~of~~ 7 MCR levee breach which ~~is~~ 8 significant safety issue.

The STP's estimation of the DBF elevation caused by a postulated MCR breach scenario is 40 ft above MSL. The breach scenario is based on the estimated breach width of 417 ft and breach time of 1.7 hours.

The NRC staff assisted by 9 ~~the~~ PNNL completed the review of the STP's MCR breach analysis in May 2011, and concluded that the applicant's analysis of the MCR breach flood is acceptable (refer to the SER Section 2.4.4). However, the author of this report (Dr. Hosung Ahn) found that the analyses of MCR breach flood by both STP and PNNL are inaccurate and non-conservative. Therefore, the author made a re-analysis of the MCR breach flood in a 10 conservative manner. This report summarizes the result of the re-analysis and findings. Section 3 of this report summarizes the regulatory requirement of dam failure analysis for reactor licensing. Sections 4 and 5 discuss the problems in STP's and PNNL's dam breach analyses, followed by the re-analysis of MCR breach flood in Section 6.

Page: 4

Number: 1 Author: d3k890 Subject: Highlight Date: 7/12/2011 5:40:40 PM -04'00'

It is true that dam failure analysis for determination of potential flood hazards is deterministic. But condition of the dam, when known, would be a factor in determining its failure during safety reviews.

Number: 2 Author: d3k890 Subject: Highlight Date: 7/12/2011 5:40:56 PM -04'00'

We assume the author means "all potential flood-causing events."

Number: 3 Author: D3B102 Subject: Comment on Text Date: 7/11/2011 5:15:11 PM -04'00'

This is related to GW DCD levels; SW DCD level is higher; i.e. not above plant grade. Therefore the statement is true but may be misunderstood.

Number: 4 Author: d3k890 Subject: Highlight Date: 7/12/2011 5:42:33 PM -04'00'

A reference in support of increased potential for MCR embankment breach because of increase in operating level from 45 to 49 ft MSL should be provided.

Number: 5 Author: D3M760 Subject: Inserted Text Date: 7/19/2011 8:07:47 PM -04'00'

How about "Raising?"

Number: 6 Author: D3B102 Subject: Replacement Text Date: 7/11/2011 3:41:04 PM -04'00'

rise

Number: 7 Author: D3M760 Subject: Inserted Text Date: 7/19/2011 8:07:47 PM -04'00'

"for a"

Number: 8 Author: D3B102 Subject: Replacement Text Date: 7/11/2011 3:42:45 PM -04'00'

could be

Number: 9 Author: D3M760 Subject: Cross-Out Date: 7/19/2011 8:07:47 PM -04'00'

Number: 10 Author: D3M760 Subject: Highlight Date: 7/22/2011 4:54:02 PM -04'00'

Implausible based on historical record...

3 Regulatory Basis

The applicable regulatory requirements for analyzing the effects of dam failures on the proposed new reactors are as follows:

- 10 CFR Part 100, as it relates to identifying and evaluating hydrological features of the site. In site evaluation, 10 CFR 100.20(c) specifies the requirement to consider physical site characteristics, including seismology, meteorology, geology, and hydrology.

- 10 CFR 100.23(d), as it sets forth the criteria to determine the siting factors for plant design bases with respect to seismically induced floods and water waves at the site.

- 10 CFR 52.79(a)(1)(iii) and GDC 2 of Part 50, as it relates to identifying hydrologic site characteristics with appropriate consideration of **the most severe** of the natural phenomena that have been historically reported for the site and surrounding area and with **sufficient margin** for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

In addition, the NRC staff has been ~~used~~ appropriate regulatory positions of the following regulatory guides for the acceptance criteria of dam failure analyses:

- RG 1.27, "Ultimate Heat Sink for Nuclear Power Plants"

- RG 1.59, "Design Basis Floods for Nuclear Power Plants," as supplemented by best current practices

- RG 1.102, "Flood Protection for Nuclear Power Plants."

Specifically, Part 100.20(c)(3) states that the **maximum probable** wind speed and precipitation must be identified and characterized in the analysis of reactor site safety. Correspondingly, the Regulatory Position 1.c in RG 1.59 specifies the requirement of analyzing dam failures to identify **the worst** site-related flood at a nuclear power plant. Section 2.4.4 of NUREG–0800 (SRP) provides technical rationale and acceptance criteria for the analysis of dam failures. In this SRP section, the terms 'conservative' is used ~~8~~ times and the term 'the most severe' is appeared ~~7~~ times in order to ~~emphasis~~ the conservatism that must be applied to the analysis of dam failures. The author of this report claims that the dam failure analyses by both STP and PNNL do not meet these regulatory requirements.

Page: 5

Number: 1 Author: d3k890 Subject: Highlight Date: 7/12/2011 5:43:41 PM -04'00'

The regulation requires "appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area" rather than "the most severe" event that can be postulated.

Number: 2 Author: D3B102 Subject: Replacement Text Date: 7/11/2011 3:43:58 PM -04'00'

using

Number: 3 Author: d3b102 Subject: Comment on Text Date: 7/11/2011 5:18:22 PM -04'00'

Not sure that this relevant to discussion on differing analyses on breach.

Number: 4 Author: D3M760 Subject: Highlight Date: 7/19/2011 8:12:15 PM -04'00'

In most of those cases, the historical context is also included. The SRP needs to state the historical context.

Number: 5 Author: D3B102 Subject: Replacement Text Date: 7/11/2011 3:44:56 PM -04'00'

eight

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seven

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emphasize

4 MCR Breach Analysis Submitted by STP

STP presented the ~~main cooling reservoir (~~MCR~~)~~ breach flood analysis in the FSAR Section 2.4.4 and the responses of relevant RAIs. The MCR is a manmade, in-ground reservoir enclosed by about 12.4 miles embankment ~~levee~~. The function of the MCR is to supply water to dissipate the heat generated from the nuclear power plants. STP analyzed the onsite flood hazard resulting from a postulated breach of the north MCR embankment ~~levee~~ located about 2,300 ft south ~~from~~ the proposed Units 3 and 4.

The MCR embankment ~~levee~~ was built with 40-feet-high rolled soil materials above the clay-sand-silt top surface. The interior face (reservoir side) of the ~~dyke~~ is lined with 2.5 ft thick soil cement blocks to absorb wave actions. The outside face of the ~~levee~~ is covered by grass on bare soils (refer to Figure 4-1). The design normal maximum operating water level in the MCR is 49 ft MSL. STP postulated the MCR ~~dyke~~ failures caused by the excessive seepage from: (1) piping through the foundations of embankment, (2) seismic activity leading to liquefaction of the foundation soils, and (3) erosion of the embankment due to overtopping or wind-wave events.

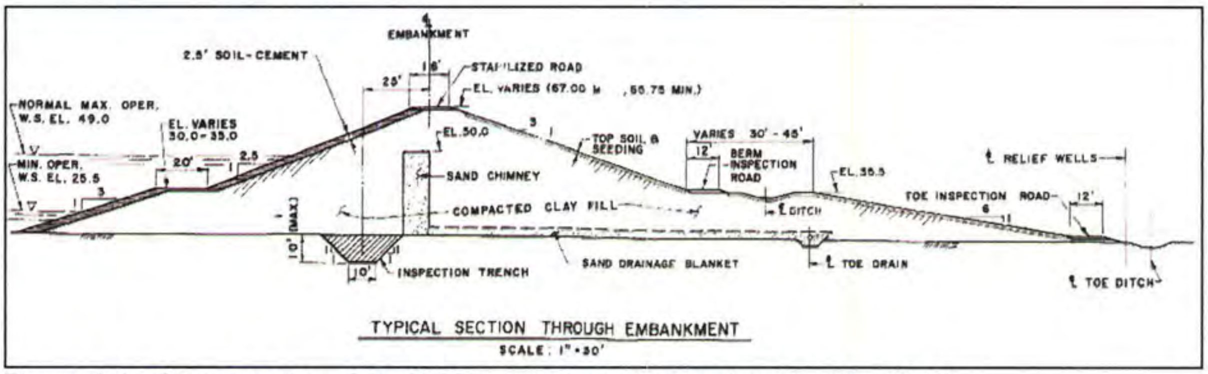

Figure 4-1. Cross section of the MCR embankment levee (provided by STP)

STP postulated and analyzed a northern MCR ~~levee~~ breach that results in the most severe flood (in terms of the flood level) at the proposed site. The following summarizes the STP's MCR breach flood analysis:

(1) STP estimated ~~levee~~ breach parameters (e.g., width and time of breach) based on ~~the~~ two empirical equations: Froelich equation for breach width and MacDonald and Langridge-Monopolis (MLM) equation for breach time. STP validated these parameter estimates using the NWS BREACH model (Fread, 1991; or just BREACH hereafter).

(2) STP simulated the MCR breach outflow hydrograph using the FLDWAV model (Fread and Lewis, 1998) with the above estimated breach parameters.

(3) STP estimates the flood characteristics (e.g., flood level, time, velocity, etc) at the plant sites using the RMA2 model (Donnell et al., 2008) with the FLDWAV breach outflow hydrograph.

Page: 6

Number: 1 Author: D3M760 Subject: Cross-Out Date: 7/19/2011 8:13:48 PM -04'00'
The specification of "MCR" has already been made.

Number: 2 Author: D3M760 Subject: Cross-Out Date: 7/19/2011 8:13:04 PM -04'00'

Number: 3 Author: d3k890 Subject: Cross-Out Date: 7/12/2011 6:12:06 PM -04'00'

Number: 4 Author: d3k890 Subject: Cross-Out Date: 7/12/2011 6:12:20 PM -04'00'

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Number: 7 Author: D3M760 Subject: Highlight Date: 7/19/2011 8:15:29 PM -04'00'
Are these the cement blocks HA includes in the roughness estimate of the breach?

Number: 8 Author: d3k890 Subject: Inserted Text Date: 7/12/2011 6:14:42 PM -04'00'
embankment

Number: 9 Author: d3b102 Subject: Cross-Out Date: 7/11/2011 5:18:37 PM -04'00'

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embankment

Number: 11 Author: D3M760 Subject: Cross-Out Date: 7/19/2011 8:16:14 PM -04'00'

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embankment

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the

Number: 15 Author: D3M760 Subject: Highlight Date: 7/19/2011 8:18:04 PM -04'00'
Is this why he mentions wind earlier?

Number: 16 Author: d3k890 Subject: Inserted Text Date: 7/12/2011 6:16:12 PM -04'00'
embankment

Number: 17 Author: d3k890 Subject: Cross-Out Date: 7/12/2011 6:17:05 PM -04'00'

Number: 18 Author: d3k890 Subject: Inserted Text Date: 7/12/2011 6:16:30 PM -04'00'
embankment

Number: 19 Author: D3M760 Subject: Highlight Date: 8/1/2011 5:48:19 PM -04'00'
From the response to RAI -02.04.04-14,

"To verify the conservatism of the selected breach parameters and the FLDWAV results, an independent, confirmatory analysis of the MCR embankment breach was performed using the BREACH model."

Specifically, STP used empirical equations to estimate dam breach parameters for earth-filled structures as recommended by the U.S. Bureau of Reclamation (USBR) (Wahl 1998). STP assumed that a service road immediately downstream of the toe of the MCR ~~dyke~~ will be the control bottom (29 ft MSL) for the postulated breach.

STP used the Froehlich equation to estimate the breach width. The estimated average breach width (e.g., top and bottom of the breach) is 417 ft. The STP estimated the time to failure of 1.7 hours using the MLM bounding equation. STP obtained the peak breach outflow of ~~62,600~~ cfs using the NWS BREACH model. They also estimated the peak breach outflow rate of 130,000 cfs using the FLDWAV model, which was used in the RMA2 model.

Evaluation of STP's Breach Analysis

It is the author's opinion that STP's breach analysis is not conservative in terms of its approach and model parameters. To the author, breach parameter estimates are of concern as they are very sensitive to the resulting DBF level. The following bullets summarize this issue:

- STP used **a mix-and-match approach** in estimating MCR breach parameters – The MLM equation for time to peak and the Froehlich equation for breach width. This results in less conservative parameters.

- In estimating the MCR breach width, STP ignored the MacDonald and Langridge-Monopolis (MLM) equation (USBR, 1998) which results in the ~~most wide~~ breach width. In a RAI response, STP stated that the MLM equation is not for breach width. This statement is incorrect. The MLM equation which was originally developed for estimating breach eroding volume has been classified and used widely in estimating breach widths (for examples, Table 1 in Wahl (1997); Wahl (2003); Gee (2010); Brunner (2007); and many others).

- STP used the Froehlich's breach width equation (1995), however USBR (1998) states that the Froehlich's equation appears to be the best predictor for the breach widths less than 164 ft because the breach width records used to develop the Froehlich equation are small. In other words, Froehlich's breach width equation is not applicable to the MCR breach case.

- The breach parameters estimated by the BREACH model is very sensitive to the roughness condition (so called the Manning's n-value) of the breach bottom. The Manning's n-value of 0.045 used in STP's BREACH modeling is not conservative. Also, STP does not consider the erosion of the levee below the 29 ft MSL in their breach analysis. Considering these two factors will substantially increase breach width and outflows as demonstrated in Section 6 of this report.

- The set-up of both STP's and PNNL's BREACH model is incorrect. In reality, the downstream tailwater condition of the MCR breach is wide 2-dimentional overland flow. However, both STP and PNNL set a limited tailwater cross-section width of 600 ft and levee crest width of 1000 ft, which restricts the simulation of breach width and outflows. This constriction results in an underestimation of the DBF flood level – It will be discuss further in Section 6 of this report.

Page: 7

Number: 1 Author: d3k890 Subject: Inserted Text Date: 7/12/2011 6:17:48 PM -04'00'
embankment

Number: 2 Author: d3k890 Subject: Inserted Text Date: 7/12/2011 6:18:28 PM -04'00'
average of

Number: 3 Author: d3k890 Subject: Inserted Text Date: 7/12/2011 6:18:35 PM -04'00'
widths

Number: 4 Author: d3k890 Subject: Highlight Date: 7/12/2011 6:19:39 PM -04'00'
Don't know what MLM "bounding equation" is.

Number: 5 Author: d3b102 Subject: Replacement Text Date: 7/11/2011 5:19:45 PM -04'00'
62,600

Number: 6 Author: D3M760 Subject: Highlight Date: 7/20/2011 9:55:54 AM -04'00'
From what HA says above, BREACH was used to validate the breach parameter results from the emperical equations, and then FLDWAV was used to provide the breach hydrograph. This statement is not consistent with that.

Number: 7 Author: D3M760 Subject: Cross-Out Date: 7/20/2011 9:56:24 AM -04'00'

Number: 8 Author: d3k890 Subject: Highlight Date: 7/12/2011 6:21:47 PM -04'00'
A reference or explanation from the author why the "parameters" are less conservative and with respect to what baseline values is needed.

Number: 9 Author: d3k890 Subject: Inserted Text Date: 7/12/2011 6:22:26 PM -04'00'
widest

Number: 10 Author: D3M760 Subject: Highlight Date: 7/20/2011 10:02:33 AM -04'00'
IF MLM computes breach erosion volumes, what assumptions are being used to estimate width?

Number: 11 Author: d3k890 Subject: Highlight Date: 7/12/2011 6:25:02 PM -04'00'
See Tony Wahl's reply to Ken See's e-mail on June 22, 2011.

Number: 12 Author: D3K890 Subject: Highlight Date: 7/13/2011 1:38:17 PM -04'00'
should this be 0.05?

Number: 13 Author: d3b102 Subject: Comment on Text Date: 7/12/2011 6:26:31 PM -04'00'
Given what the Manning's n is used for and is meant to represents, this is a reasonable and conservative value. A reference for justification of the author's statement is needed.

Number: 14 Author: d3b102 Subject: Comment on Text Date: 7/11/2011 5:25:06 PM -04'00'
This statement and conclusion should be included in section 6 rather than here.

Number: 15 Author: d3b102 Subject: Comment on Text Date: 7/12/2011 6:27:34 PM -04'00'
This statement and conclusion should be included in section 6 rather than here.

5 MCR Breach Analysis by PNNL

PNNL performed an independent confirmatory analysis of the MCR breach flood using the breach models developed by the applicant. The following summarizes the procedure used by PNNL:

(1) PNNL simulated breach widths and outflow hydrographs using the BREACH model with three Manning's n-values, then picked the most conservative one as a postulated breach scenario.

(2) They [1]confirmed the breach scenario using empirical breach parameter equations as well as an actual breach case from the Martin Cooling Pond in Florida.

(3) PNNL used the applicant-provided RMA2 model to simulate flood ~~characters~~ [2] the site.

PNNL carried out a sensitivity analysis by varying some of the BREACH model parameters to determine the sensitivity of model input parameters (e.g., levee geometry, embankment material properties, headwater/tailwater conditions, etc.) on the breach parameters (e.g., width, time, peak flow) (PNNL, 2011). As a result of the BREACH sensitivity analysis, [3]PNNL identified that the Manning's n-value that represents the bottom roughness condition is very sensitive to the breach parameters and ~~outflows~~. [4]They used the following three different n-values in the BREACH simulation:

- Simulation 1: n = 0.025
- Simulation 2: n = 0.050
- Simulation 3: n = 0.075.

Then, PNNL selected [5]~~the~~ Simulation 3 as the postulated breach scenario as it produces the largest breach peak outflow (130,000 cfs). [6]The corresponding peak breach width and time are 515 ft and 1.99 hours.

PNNL also compared the BREACH-simulated outflow characteristics (peak flow and time, breach width) with the selected historical data from the database prepared by USBR (1998). From the review of the recorded breach events, PNNL ~~selected~~ [7]recorded breach event occurred in 1979 at the Martin Cooling Pond in Florida. The reservoir condition is ~~similar~~ [8] that of the MCR in terms of storage volume and water depth. [9]The final average breach widths for the Martin Cooling Pond breach and the estimated MCR breach are 607 ft and 688 ft, respectively. Based on this comparison, PNNL concluded that their MCR breach parameters and flood estimates are reasonable.

Evaluation of PNNL's Breach Analysis

[10]However, PNNL does not justify the selection of the n-value of 0.075 or why this value is conservative. [11]Further, the author of this study found that PNNL's BREACH model to estimate the breach outflows and the sensitivity analysis is flawed by an incorrect model set-up. In

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Number: 1 Author: d3b102 Subject: Comment on Text Date: 7/11/2011 5:27:18 PM -04'00'
compared would be a more accurate word

Number: 2 Author: d3k890 Subject: Inserted Text Date: 7/12/2011 6:28:34 PM -04'00'
characteristics

Number: 3 Author: D3M760 Subject: Highlight Date: 7/20/2011 10:30:02 AM -04'00'
It is unclear what HA is saying.

Number: 4 Author: d3k890 Subject: Inserted Text Date: 7/12/2011 6:38:14 PM -04'00'
discharge

Number: 5 Author: D3M760 Subject: Cross-Out Date: 7/20/2011 10:30:30 AM -04'00'

Number: 6 Author: d3b102 Subject: Replacement Text Date: 7/11/2011 5:29:24 PM -04'00'
which they determined was reasonable and conservative.

Number: 7 Author: d3b102 Subject: Replacement Text Date: 7/11/2011 5:30:15 PM -04'00'
found

Number: 8 Author: d3b102 Subject: Replacement Text Date: 7/11/2011 5:31:12 PM -04'00'
most similar

Number: 9 Author: d3b102 Subject: Replacement Text Date: 7/12/2011 6:39:23 PM -04'00'
which have been found to be reservoir parameters correlated to breach flow and width.

Number: 10 Author: d3b102 Subject: Comment on Text Date: 7/12/2011 6:45:02 PM -04'00'
A realistic range was examined. Higher values found in literature reflect additional roughness to heavily vegetated environments, winding channels,etc rather than relatively quickly eroding sediment material from a primarily straight channel like what would be found in an embankment breach. Even in natural channels a Manning's n or 0.05 would be considered substantially rough.

Number: 11 Author: D3M760 Subject: Highlight Date: 7/20/2011 4:57:26 PM -04'00'
I think he included the concrete bricks from the upstream in his estimate of breach roughness.

addition, validating the BREACH simulating result by a single arbitrarily selected breach record is not objective. The following summarizes the author's main concerns on the PNNL's analysis:

- PNNL's calculation package (2011: calpack) provides breach parameter estimates using three empirical equations (namely USBR, MLM, Froehlich) and the BREACH model. However, PNNL does not explain why MLM estimates that are the most conservative are excluded in further breach analysis. This is against general engineering practice or the guidance by federal agencies. Both USBR (1998) and USACE (2007) recommend using all available methods to determine breach parameters as the process involves a lot of uncertainty.

- PNNL used a faulty BREACH model setup by using a limited levee crest width of 1000 ft and tailwater bottom width of 600 ft (refer to Figure 5-1), which constrict the breach process and the resulting breach outflows. Finally, this modeling error results in an underestimation of breach width, outflows, and DBF flood level. This is an unacceptable modeling error. *(Note: The author notified this error to PNNL and NRC staffs in advance (February, 2011) of completing the SER, but my input has been ignored)*.

- The NWS BREACH model is based on the Meyer-Peter & Muller (MPM) sedimentation equation which results in a lowest sedimentation rate compared to other methods (refer to "Erosion and Sedimentation", by P. Julien, 2010). Many hydrologists pointed out that the MPM method is not adequate for dam breach conditions as it is developed for small mountain creeks.

- Because the result of BREACH model is highly ~~depending~~ on the model inputs, many hydrologists discredited the use of the BREACH model (e.g., Wahl, 1999). Also, the author (Dr. Fread) of the BREACH model recommends that BREACH model be used supplemental purpose only. PNNL's approach that relies only on the BREACH could be biased as this report demonstrated.

- Both STP and PNNL ignored the effects of sand core and clay cement blocks on the BREACH mode. Also they ignored the effects of scour hole on the breach process. Proper consideration of these factors will increase breach outflow and DBF level estimates substantially ~~this report will be demonstrated~~.

The next section presents a re-analysis of MCR breach flood by considering the above concerns and by adding a reasonable conservatism on the estimation.

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Number: 1 Author: d3b102 Subject: Comment on Text Date: 7/12/2011 6:48:15 PM -04'00'

The determination of a historical and similar breach case was not arbitrary and it was objective.

The reservoir volume and depth of the water prior to breaching are typical parameters used for estimating breach parameters (peak flow, timing and width). PNNL used the MCR characteristics to select the best historical case to compare to. This was not arbitrary or subjective. This selection yielded only one case for the reasonable combination of reservoir parameters used.

Number: 2 Author: D3M760 Subject: Highlight Date: 7/22/2011 5:17:50 PM -04'00'

PNNL was directed to confirm if the applicant's analysis was reasonable and conservative.

Number: 3 Author: d3b102 Subject: Comment on Text Date: 7/12/2011 6:49:43 PM -04'00'

The PNNL results demonstrated that reasonable and conservative parameter values yield results that are similar to the applicants and are similar to historical observations within a database suggested by the NRC.

Number: 4 Author: d3b102 Subject: Comment on Text Date: 7/12/2011 6:50:06 PM -04'00'

This statement should be supported with some addtional references.

Number: 5 Author: D3K890 Subject: Highlight Date: 7/13/2011 1:47:19 PM -04'00'

Number: 6 Author: d3b102 Subject: Comment on Text Date: 7/11/2011 6:06:57 PM -04'00'

Wahl(2004):

"The results from use of the physically
based NWS-BREACH model were reassuring because they
fell within the range of values obtained from the regression-based
methods. However, at the same time, they also helped to show
that even physically based methods can be highly sensitive to the
assumptions of the analyst regarding breach morphology and the
location of initial breach development. The NWS-BREACH
simulations demonstrated the possibility for limiting failure mechanics
that were not revealed by the regression-based methods."

Number: 7 Author: D3M760 Subject: Inserted Text Date: 7/20/2011 4:59:38 PM -04'00'

"dependent"

Number: 8 Author: D3M760 Subject: Highlight Date: 7/20/2011 5:09:03 PM -04'00'

Reference?

Number: 9 Author: d3b102 Subject: Comment on Text Date: 7/12/2011 6:53:59 PM -04'00'

PNNL conducted sensitivity analyses on parameters that represented the strength of the embankment material. Sand, a weak material, was represented within PNNL analysis. PNNL treated the embankment as if it was of uniform strength. These analyses included values of embankment material strength consistent with sand. In sum an embankment comprised entirely of sand represents a conservative assumption.

Number: 10 Author: d3b102 Subject: Replacement Text Date: 7/12/2011 9:35:46 AM -04'00'

as this report will demonstrate.

(a) PNNL's BREACH Input File for Simulation 3

```
MCR
50.9     66.      29.      32.      0.       0.       .02      .8
30.      30.      30.      0.       0.       0.       0.       0.
0.       1.       100.     0.       0.       0.       0.       0.
7050.    7000.    6850.    5690.    2050.    0.       0.       0.
52.      49.      29.      25.5     20.      0.       0.       0.
28.      29.      32.      34.      40.      45.      50.      0.
0.       600.     1200.    1600.    2000.    2400.    2800.    0.
.06      .06      .06      .06      .06      .06      .06      0.
2.5      3.       0.       0.       0.       0.       0.
0.       0.       0.       0.       0.       0.       0.
0.001    .35      105.     .075     15.      200.     8.
1.       16.      1000.    8.       0.       0.       0.       0.
.001     0.001    .1       120.     .01      1.       0.
```

This small bottom tailwater section (600 ft) constricts the simulated breach width and outflows.

Again, this small levee crest length constricts the breach width and outflows.

Manning's n-value

(b) Hosung Ahn's BREACH Input File for Simulation 3

```
MCR Ahn's Run
50.9     66.      29.      32.      0.       0.       .02      .8
30.      30.      30.      0.       0.       0.       0.       0.
0.       1.       100.     0.       0.       0.       0.       0.
7050.    7000.    6850.    5690.    2050.    0.       0.       0.
52.      49.      29.      25.5     20.      0.       0.       0.
28.      29.      32.      34.      40.      45.      50.      0.
0.       3000.    3000.    3000.    3000.    3000.    3000.    0.
.06      .06      .06      .06      .06      .06      .06      0.
2.5      3.       0.       0.       0.       0.       0.
0.       0.       0.       0.       0.       0.       0.
0.001    .35      105.     .075     15.      200.     8.
1.       16.      3000.    8.       0.       0.       0.       0.
.001     0.001    .1       120.     .01      1.       0.
```

Figure 5-1. Comparison of PNNL's BREACH input file and a corrected one by the author.

This page contains no comments

6 Re-analysis of MCR Breach Flood

This section presents the procedure, result, and findings of the MCR breach flood re-analysis which was performed to support the concerns raised by the author. The re-analysis is based on a comprehensive modeling approach. The author considered all applicable empirical breach equations and multiple breach models. As was done by the STP, the re-analysis used a deterministic dam breach analysis that does not consider the condition of the MCR ~~levee~~ [1]r the frequency of the potential MCR ~~levee~~ [2]ilures. The author postulated multiple breach scenarios in a conservative manner. The errors found in STP's and PNNL's BREACH modeling were fixed here. This study also considers the effects of sand core, cement blocks, and scour hole on the breach progression and the resulting flood.

Because there is no model that can handle both breach and flood simulation simultaneously, the author adopted a multi-step approach in series. The procedure used in the re-analysis is summarized as follow~~ing~~:[3]

(1) This study includes an estimation of breach parameters using all applicable empirical equations in a comprehensive manner. The breach parameters include width and time of breach as well as peak breach outflows.

(2) The author used two physically-based breach models, namely NWS BREACH (or just BREACH hereafter) and FLDWAV, to obtain and validate the breach parameters estimated by empirical equations and to generate breach outflow hydrographs.

(3) The author used the FLO-2D model to simulate the overland flow at the proposed plant site using the FLO-2D hydrodynamic model.

[4]ecause the RMA2 model which was used by STP that is not available to the NRC staff, the author used the FLO-2D model which is another numerical 2-dimentional hydrodynamic flow routing software. The following figure compares the different modeling approaches used for the MCR breach flood analysis. The following subsections describe details of each step in the re-analysis.

STP's Approach
Empirical Equ's → Breach Width and Time → FLDWAV → Hydrograph → RMA2
Use BREACH to validate width and time

PNNL's Approach
BREACH --→ Hydrograph → RMA2

Re-analysis Approach
Empirical Equ's → Breach Width and Time → FLDWAV → Hydrograph → FLO-2D
BREACH--→ Hydrograph → FLO-2D

Figure 6-1. Comparison of MCR breach analyses by STP, PNNL, and the author.

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Number: 1 Author: d3k890 Subject: Inserted Text Date: 7/12/2011 6:55:36 PM -04'00'
embankment

Number: 2 Author: d3k890 Subject: Inserted Text Date: 7/12/2011 6:55:52 PM -04'00'
embankment

Number: 3 Author: d3k890 Subject: Inserted Text Date: 7/12/2011 6:56:27 PM -04'00'
s

Number: 4 Author: d3b102 Subject: Comment on Text Date: 7/12/2011 6:58:12 PM -04'00'
RMA2 is distributed by USACE; I believe it is free. The full SMS software is a licensed software which is used as a GUI to support RMA2 analysis and other modeling software. A demonstration copy of the SMS is available for free. RMA2 input files were supplied by applicant to PNNL via NRC. PNNL modified the applicant's RMA2 input files to carry out its confirmatory analysis.

6.1 Concerns on the MCR Breach Analysis

In general, dam breach parameters are very sensitive on the resulting flood levels. This is true for the MCR breach case as shown by both STP's and PNNL's studies. At the beginning of the STP COL license application, STP postulated a very conservative breach scenario that results in a conservative design basis flood (DBF) level. This initial breach scenario is even more conservative than that for Units 1&2. However, STP changed the original scenario by adopting a narrow breach width through the FSAR Revision 2. The table below summarizes the different breach scenarios.

Table 6.1-1. Comparison of different breach scenarios and resulting design basis flood (DBF) levels.

Source	Distance from levee to the site (ft)	Initial Reservoir Water Level (ft MSL)	Breach Width (ft)	DBF Level at the site (ft MSL)
UFSAR for Units 1 & 2	600	50.9	2000	50.8
FSAR Rev. 0 for Units 3 & 4	2300	50.9	4757	47.6
FSAR Rev. 2 for Units 3 & 4	2300	50.9	417	40
Victoria ESP	1000	93.9 ft NAVD88	2034	90.6 ft NAVD88 (~10 ft in depth)

Note: The estimated DBF levels for the STP FSAR Rev. 0 andVictoria ESP is based on the simulation of the Delft3D model, while that of STP Rev. 2 is by the RMA2 model.

The change of the STP's breach scenario triggered ~~an~~ attention to the NRC reviewers, resulting in numerous RAIs, site audits, and independent confirmatory analyses. However, the author believes that this issue has not been resolved in the SER in a satisfactory manner. It is the author's opinion that both STP's and PNNL's breach flood estimates are neither correct nor conservative. To help resolv~~ing~~ this issue, the author performed a comprehensive re-analysis.

Specifically, it is my opinion that both STP's and PNNL's approaches are biased by using selective methods and by ignoring conservative equations or methods without proper justification. In general, it is recommended by many papers and federal guides that the engineers who practice dam breach analysis should consider a variety of methods as dam breach estimations contain a considerable uncertainty. For instance, Brunner (2007) in USACE provides the following recommendation:

> "In general, several methods should be used to predict a range of breach sizes and failure times. It is recommended that the user uses all empirical equations, such as the three recommended above: Froehlich (1995b), MacDonald and Langridge- Monopolis (1984), and Von Thun and Gillette (1990). Additional regression equations can also be used, as well as any of the physically based computer models (the BREACH model is currently recommended). This will lead to a range of values for the breach size and

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Number: 1 Author: d3k890 Subject: Highlight Date: 7/12/2011 7:02:35 PM -04'00'

We assume the author means that estimated flood water surface elevations are very sensitive to dam breach parameters because estimated dam breach discharge hydrograph is very sensitive to selected dam breach parameters.

Number: 2 Author: d3b102 Subject: Replacement Text Date: 7/12/2011 9:50:28 AM -04'00'

the

Number: 3 Author: d3k890 Subject: Highlight Date: 7/12/2011 7:27:14 PM -04'00'

At the time STP submitted their revised MCR embankment breach and subsequent flooding analysis to use RMA2 with a much narrower breach, PNNL's discussions with NRC determined that the applicant's design-basis analysis must be reviewed to determine that it was correct, reasonable, and conservative. Although the applicant's FSAR Rev 0 analysis was more conservative than the revised one, PNNL's responsibility was to review the applicant's revised submittal. PNNL's understanding is that the NRC safety review process allows the applicant to use site-specific analyses supported and justified by site-specific data to perform a reasonable and conservative analysis. PNNL also understand that the NRC safety review does not require postulation of an absolute worst-case scenario which would be unreasonably conservative for the site.

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e

Number: 5 Author: d3b102 Subject: Highlight Date: 7/12/2011 9:52:23 AM -04'00' failure times. A sensitivity analysis of breach parameters and times should be performed by running all of the parameter estimates."

Because the breaching process is complex and uncertain, the HEC-RAS manual (2008) suggests that the modeler should try to come up with several estimates of breach parameters and breach sizes, and then put together a matrix of potential breach sizes and times. One example would be use of two different sets of regression equations and one of the breach simulation models to estimate the breach parameters. Hydrologic Engineering Center in Davis California performed several dam breach studies using both the Froelich (1995) and the MLM regression equations, as well as the BREACH model (Fread, 1988). They noticed that all three methods give different answers for the breach dimensions and time. They ran each of these estimates as a separate trial within HEC-RAS to make reliable breach flood analysis.

Therefore, the author tried to use all available empirical breach equations and models, from which conservative breach scenarios are postulated in compliance with the GDC 2 requirement.

Number: 1 Author: d3b102 Subject: Comment on Text Date: 7/12/2011 9:55:21 AM -04'00'

Applicant's analysis used several methods for estimation of breach parameters; PNNL analysis showed that they used reasonable methods and made conservative parameter selections.

6.2 Assumptions Used in Postulating MCR Breach Scenarios

The author used the following assumptions to postulate MCR breach scenarios:

(1) The potential causes of the MCR breach are one or combination of the following three components:

- a piping failure caused by either seepage through the ~~levee~~ its foundation,
- liquefaction of the foundation soils, or
- ~~levee~~ erosion from rain, flood, or others.

(2) The initial MCR water ~~bevel~~ 50.8 ft MSL. The initial tailwater condition is dry – There is no rain or flood before the breach. STP determined the initial headwater level by considering the normal maximum MCR operating level of 49 ft MSL plus a margin for wind setup in the reservoir.

(3) A postulated MCR ~~levee~~ breach occurs at the nearest MCR ~~levee~~ section from the units 3 or 4 to maximize the DBF level. The exact placement of the breach section in this study was determined based on the FLO-2D model grid.

(4) Other than the Manning's roughness coefficient (n-value) used in the BREACH model, the conditions and parameter values of the embankment materials are the same as those used by STP.

(5) The author used the STP's BREACH model setup, but ~~levee~~ crest width and tailwater cross-section were corrected to match the field condition.

(6) The breach scenarios include the effects of scour hole, sand chimney and blanket, and cement blocks on the breach process.

By definition, dam is a wall built across a river or stream to stop water from flowing, whereas levee (or dike) is a wall used to manage or prevent water flow into specific land regions. Dams are commonly built on a rigid bottom and side foundation for the stability reasons, however most levees are build on ~~a~~ bare soils. Therefore, the mode of breach and parameters for levees (especially for long levees) could be different from those for dams as:

- Levee breach tends to be wider than dam breach, if the stored water in the reservoir is sufficient in quantity and head
- Levee breach accompanies a deep scour hole
- Tailwater in the breach downstream is characterized by a two-dimensional overland flow whereas dam breach flood is a steep channel flow.

The above factors were considered in this study. As the breach bottom roughness coefficient and scoring hole are critical to determining the beach process and resulting outflows, the following subsections focus on the discussion of the maximum levee breach width and ~~score~~ hole created during the levee breach.

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embankment

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embankment

Number: 3 Author: D3K890 Subject: Inserted Text Date: 7/13/2011 1:56:56 PM -04'00'
level

Number: 4 Author: d3k890 Subject: Inserted Text Date: 7/12/2011 7:30:05 PM -04'00'
embankment

Number: 5 Author: d3k890 Subject: Inserted Text Date: 7/12/2011 7:30:13 PM -04'00'
embankment

Number: 6 Author: D3K890 Subject: Highlight Date: 7/13/2011 1:59:19 PM -04'00'

Number: 7 Author: d3k890 Subject: Highlight Date: 7/12/2011 7:30:51 PM -04'00'

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embankment

Number: 9 Author: d3k890 Subject: Cross-Out Date: 7/12/2011 7:31:49 PM -04'00'

Number: 10 Author: D3M760 Subject: Highlight Date: 7/22/2011 5:30:45 PM -04'00'
The tailwater cross section is defined by the downstream end of the breach not the infinitely wide valley. The purpose of the tailwater width in BREACH is for backwater effects on breach formation. His approach using 2D flow with an infinitely wide valley has removed the backwater effect altogether.

Number: 11 Author: d3b102 Subject: Replacement Text Date: 7/12/2011 9:59:22 AM -04'00'
scour

6.2.1 Potential Maximum Levee Breach Width

Some hydrologists assume that there is a physically-acceptable maximum breach width regardless of reservoir-levee conditions. For instance, Von Thun and Gillette (1990) states in their unpublished paper that "the small database of large-dam failures tends to indicate 500 ft as a possible upper bound for breach width." However, this assumption is not true if we consider actual dam failure records. There are numerous reported cases where breach widths for dams and levees exceed far beyond 500 ft. The [1]~~levee~~ breach of the Martin Cooling Pond in Florida in 1979 is an example where the recorded breach width ended up to 600 ft. STP has been arguing that the MCR breach width of 417 ft is credible considering the historical data, such as the Teton dam failure case. Therefore, the author reviewed the historical breach records to determine the reasonable MCR ~~levee~~ [2]reach width.

Breach Formation Factor

There are many factors that govern the breach process. MacDonald and Langridge- Monopolis (1984) found that the product of the breach outflow volume and the difference in elevation of the peak and base reservoir levels (e.g., V_w x h_w) is a reasonable variable for predicting breach eroded volume and breach width. They called this variable (V_w x h_w) [3]~~as~~ the breach formation factor. Their finding indicates that the breach process will be continued if there are sufficient water storage and head, or if there is no external factors that interrupt the breach process (e.g., hard rock bottom or abutment, structures, etc). Dams that have very large volumes of water and have long dam crest lengths will continue to erode for a long duration (Gee, 2008). That is, as long as a significant amount of water is flowing through the breach, there will be wider breach width and longer time. This ~~continued~~ [4]each ~~are~~ [5]so found ~~from~~ [6]e BREACH and FLDWAV simulations (~~It~~ [7]ill be shown later).

Historical Maximum Breach Widths

The author reviewed the USBR dam breach database (Wahl, 1998) to see the maximum breach width that occurred historically. This database has over 108 historical dam breach cases from the world with 410 breach records - There are multiple breach estimation records for a breach in many cases. From this database, the recorded maximum dam breach width of [8]800 ft was observed from the [9]achhu II dam failure in India. Excluding the Machhu II case, the maximum dam breach width is [11]0 ft, which is followed by [12]8 ft, 610ft, 551 ft, and so on. [10]ese data indicate that the recorded dam breach widths are wider than the STP's postulated MCR breach width of 417 ft.

There are [13]any dam breach database [14]ailable, however these databases do not include levee breach data or there is no known levee breach database. In the meantime, some reports or papers provide limited levee breach data for selective levee system(s) or region(s). For instance, [15]agy (2006) provides 39 levee failure cases in the Eastern Europe Rivers, where

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Number: 1 Author: d3k890 Subject: Cross-Out Date: 7/12/2011 7:35:58 PM -04'00'

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embankment

Number: 3 Author: d3k890 Subject: Cross-Out Date: 7/12/2011 7:36:28 PM -04'00'

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continuation of

Number: 5 Author: d3k890 Subject: Inserted Text Date: 7/12/2011 7:37:26 PM -04'00'
was

Number: 6 Author: d3k890 Subject: Inserted Text Date: 7/12/2011 7:37:52 PM -04'00'
in

Number: 7 Author: d3b102 Subject: Replacement Text Date: 7/12/2011 10:01:56 AM -04'00'
as

Number: 8 Author: D3K890 Subject: Highlight Date: 7/13/2011 6:36:21 PM -04'00'
The Reclamation dam breach database lists observed characteristics of breach events from multiple sources and also lists which of these is the "best reliable information." For the Machhu II, India (1979) failure, one of the sources reported an average breach width of 1768 m (5800 ft). However, the "best reliable information" for the average breach width for Machhu II event is listed as 540 m (1772 ft) in the Reclamation database.

Number: 9 Author: d3b102 Subject: Comment on Text Date: 7/12/2011 10:03:28 AM -04'00'
Because this is an extreme case we should further review that conditions that led to this failure and breach geometry.

Number: 10 Author: D3K890 Subject: Highlight Date: 7/14/2011 1:06:58 PM -04'00'
The last three historical breach events indeed have a wider dam breach width than that postulated by STP. PNNL's review and confirmatory analysis used the Martin Cooling Pond breach event with 610 ft width, which was found to be most similar to the MCR embankment scenario.

Number: 11 Author: D3K890 Subject: Highlight Date: 7/13/2011 6:14:03 PM -04'00'
PNNL's review of Reclamation dam breach database did not identify a breach event with 820 ft width.

Number: 12 Author: D3K890 Subject: Highlight Date: 7/13/2011 6:22:59 PM -04'00'
According to the Reclamation dam breach database, Swift, Montana (1964) failure had an average breach width of 225 m (738 ft); Martin Cooling Pond, Florida (1979) failure had an average breach width of 186 m (610 ft); and Salles Oliveira, Brazil (1977) had an average breach width of 168 m (551 ft). We assume that the aothor is referring to these historical breach events.

Number: 13 Author: D3K890 Subject: Highlight Date: 7/14/2011 5:43:12 PM -04'00'
We are aware of two dam breach databases -- the Reclamation dam breach database (Wahl, 1998) and Pierce (2008) database -- not "many."

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[1]000 ft is the maximum levee breach width. This maximum is followed by breach width of 1300 ft for the 2nd, 1000 ft for 3rd, and so on.

The URS (2007) report provided 14 historical levee breach cases from the [2]alifornia Delta Levee system, where 1018 ft is the maximum. This maximum is followed by breach width of 950 ft for 2nd, 926 ft for 3rd, and so on.

[3]rom these levee breach records, we can conclude that levee breach widths are generally wider than those of dams and that potential levee breach widths could be wider than 1000 ft. This conclusion supports the use of the MLM breach width equation for the MCR breach scenario.

6.2.2 Breach Scour Hole

Scour hole is not a common phenomenon in dam breaches because most dams are constructed on a hard rock foundation. That is why there are not many references that address the scour hole issue in dam breach. However, the scour hole is very common and an important issue in levee breach. The scour hole could increases the capacity (area) of breach outflows significantly and thus for flood levels. Some references address this specific issue (SFWMD, 1980; Zhang and Wang, 2001; Nagy, 2006; etc.).

Scour hole could ~~be occurred~~ [6]ght below the levee (Figure 6.2-1) and/or it could [4]~~be happened~~ [5] at the headwater or tailwater zones or both (SFWMD, 1980; Figure 6.2-2 below). Nagy (2006) pointed out that developing a scour hole (pit) depends on subsurface (foundation) conditions as well as the mechanism of levee destruction. In terms of subsurface condition, scour holes are highly likely if the subsoil below levee is composed of grainy and transitional soils. The width of scour hole is usually smaller than the breach width ~~if~~ f[7]undation is more compacted or less erodible than the levee itself.

SFWMD (1980) reported that the levee breach of the Martin[8] Cooling Pond in Florida in October 1979 created a big scour hole having the dimension of:

- width of 450 ft
- length of 700 ft
- depth of 29 ft from the land surface (or the bottom of the levee), and
- Final levee breach width of 600 ft.

The scour hole for the Martine Cooling Pont has [9]U shape across the flow direction, having a wide top width but narrow bottom width (about 190 ft wide and 400 ft long). The maximum depth ~~was appeared at~~ [10]ght [11]wnstream ~~from~~ [12] levee.

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It is not surprising that levees experience wider breaches compared to embankment dams used for static storage. Levees are generally used to contain discharges within stream channels and fail by overtopping during floods larger than their design basis. When large floods cause these levee failures, the flood overtops the levee over significantly long distances along the levee's length. The duration of overtopping can also be significantly long depending on the magnitude of the flood compared to the levee's design basis. The breached levee also experiences substantial shear forces from the flood discharge. These conditions promote a longer levee breach.

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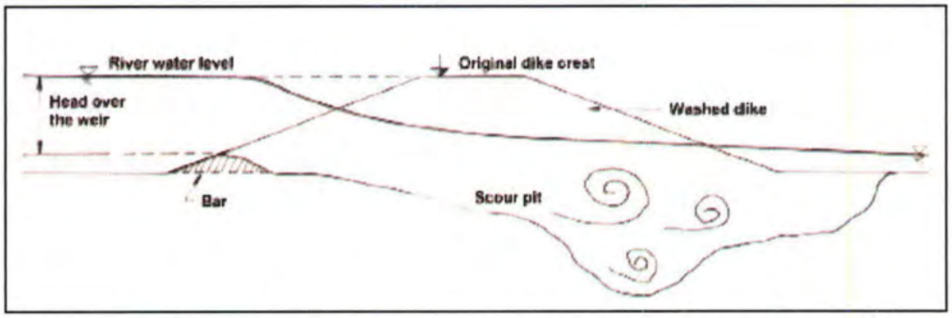

Figure 6.2-1. Typical cross section of a levee failure with scour hole (pit) (after Nagy, 2006).

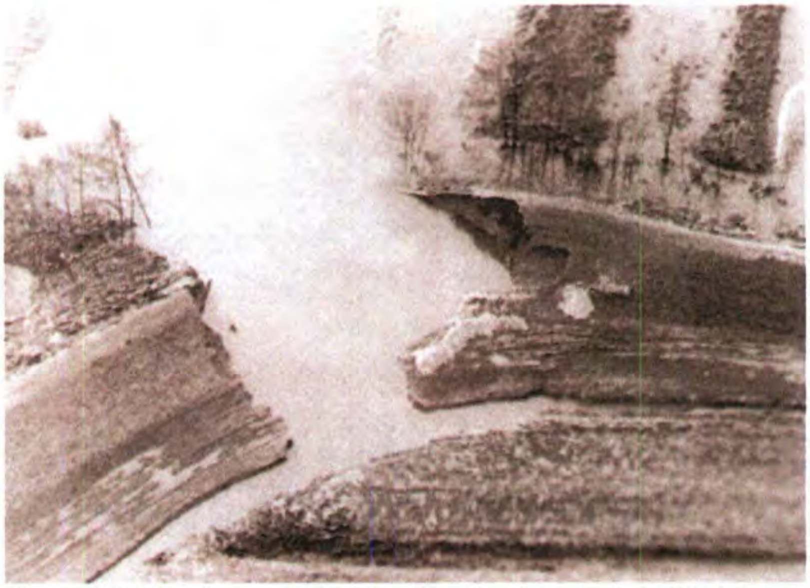

Figure 6.2-2. ~~Potograph~~ of Elba River bank breach in 1976 (Nagy, 2006)

Zhang and Wang (2001) simulated the depths of scour hole from levee breach using a numerical model. The model is based on a solution of 2-dimentional shallow water and suspended sediment equations. They estimated that, under the breach width of 328 ft and the median grain side of 0.3 mm, the maximum scour hole depth from the surface is estimated to be about 20 feet. Their study indicates that the maximum scour hole occurs at the tailwater portion (e.g., after the toe) of levee or about 100 to 165 meters downstream from the invert of breach. They found that the hole ~~occurs at the~~ early ~~stage of~~ breach (1.7~10 minutes).

Based on the result of Zhang and Wang (2001), STP postulated a scour hole from the MCR breach at the toe of ~~levee~~. This hole has width of 380 ft, length of 203 ft, depth of 20 ft from the surface elevation 29 ft MSL. They used the hole in estimating the volume of eroded materials to evaluate the effects of sedimentation deposition at the site on the DBF flood level. However, STP assumed no scour hole underneath the MCR ~~levee~~ thus no impacts of the hole on the MCR breach process and outflows. They ~~justified~~ that the road right after the levee prevent scour hole below the ~~levee~~ without proper justificaiton. STP's ignoring the scour hole in the MCR ~~breach~~ results in significant underestmation of the DBF level as my analysis shown later. This is a significant drawback in terms of a site safety analysis.

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Number: 6	Author: D3K890	Subject: Inserted Text	Date: 7/14/2011 7:01:39 PM -04'00'	the embankment
Number: 7	Author: D3K890	Subject: Inserted Text	Date: 7/14/2011 7:01:52 PM -04'00'	a
Number: 8	Author: D3K890	Subject: Inserted Text	Date: 7/14/2011 7:01:59 PM -04'00'	a
Number: 9	Author: D3K890	Subject: Inserted Text	Date: 7/14/2011 7:02:07 PM -04'00'	and a
Number: 10	Author: D3K890	Subject: Inserted Text	Date: 7/14/2011 7:02:37 PM -04'00'	embankment
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Number: 16	Author: D3K890	Subject: Inserted Text	Date: 7/14/2011 7:05:26 PM -04'00'	formation right underneath
Number: 17	Author: D3K890	Subject: Inserted Text	Date: 7/14/2011 7:05:46 PM -04'00'	embankment
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The author considered the following three cases to consider the scour hole effects in MCR breach:

- Case 1: No scour hole under the levee as in the case of the STP scenario
- Case 2: Extend the scour hole to the levee below, with an average hole depth of 10 ft (i.e., 0 ft on the invert and 20 ft at the toe of levee)
- Case 3: Extend the scour hole to the levee below, with an average hole depth of 20 ft (i.e., an average of 20 ft depths at both invert and toe of levee)

The pictorial descriptions of these cases are shown in the following figure. This study considers the above three cases in the breach flood analysis later.

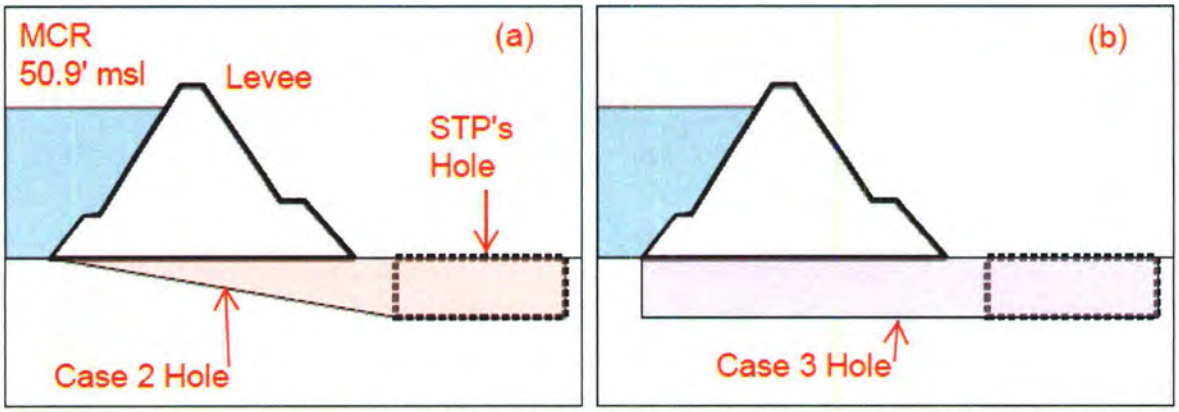

Figure 6.2-3. Schematic plots of MCR breach scenarios with different scour holes: (a) case 2 with 10 ft hole depth, (b) case 3 with 20 ft hole depth (not to scale).

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6.3 Breach Parameter Estimation Using Empirical Equations

6.3.1 Dam Breach Analysis

In general, dam breach analysis includes dam breach simulation and downstream flood routing. The analysis uses either empirical regression equations or numerical models, or both. Because there is no model that can perform the two simulations together, hydrologists use a multi-step approach. For instance, breach parameters are estimated using empirical equations and the~~m~~ provided ~~the parameters~~ as input to breach outflow and flood routing models. The breach parameters include width (B) and time (Tf) of breach, rate (Qp) and time (Tp) of peak outflow, and breach outflow hydrograph (Q(t)) (refer to Figure 6.3-1). Several methods are available for estimating breach parameters (Wahl (1998):

- Simple regression equations to predict breach width and time
- Simple regression equations to predict peak outflows
- Physically-based breach simulation models, such as BREACH, FLDWAV, HEC-RAS, FLO-2D, etc.

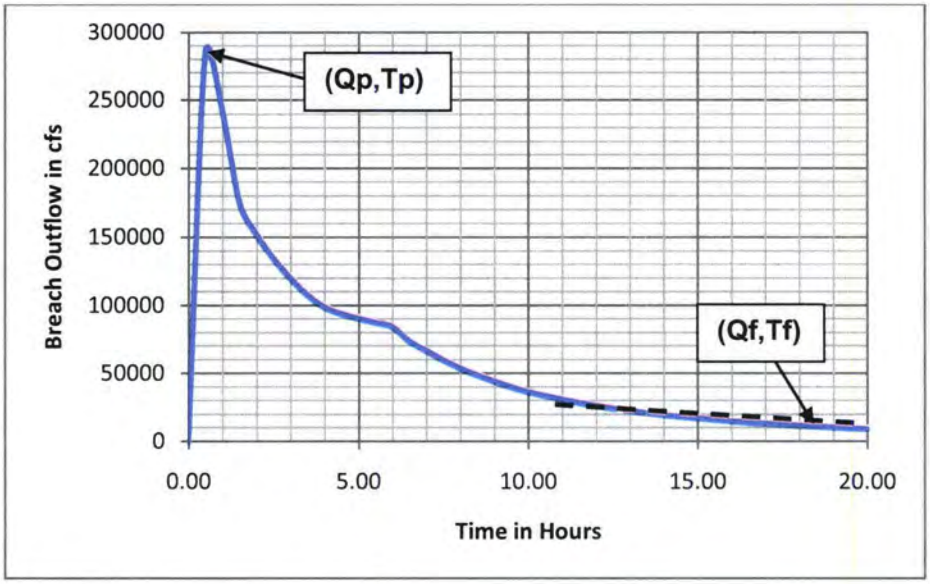

Figure 6.3-1 An example of breach outflow hydrograph simulated by the FLDWAV model for the breach width of 500 ft and the time to peak of 0.5 hours.

Regression equations estimates breach parameters as functions of one of more dam and reservoir parameters, such as storage volume (V_w), depth of water of ~~depth~~ breach (h_w), etc. The equations are based on regression analyses of data collected from actual dam failures. Several federal agencies, such as USBR, provide the database of historical dam failures.

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at time

However, these databases are relatively lacking in data from failure of large dams, with about 75 percent of the case having a dam height less than 50 ft (Wahl, 1998). Further, they do not include levee failure records.

Chauhan et al. (2004) concluded, based on a simulation of DAMBRK (Fred,1984) and an empirical dam breach equation, that the use of breach parameters estimated from empirical equations as inputs to physical breach models such as DAMBRK typically results in significant overestimation of peak breach flows. However, their case study uses very limited breach conditions – ~~It~~ could end ~~up~~ different conclusion if one use different equations, models, and breach conditions. In fact, many dam breach simulation studies (e.g., Gee, 2008; Gee, 2010; Wahl, 1997, etc.) show that physical models give either higher or lower peak flows depending on site conditions.

As my study demonstrated later, physical models, like BREACH or FLDWAV, tend to produce overestimation of peak flow and time. It is my opinion that the breach parameter estimates using empirical equations are more credible than those by physical model because (1) empirical equations are based on real data, (2) validations of physical models are usually done with limited number of historical records or no recorded at all for future breach prediction, (3) breach parameter estimates using physical models ~~are~~ widely vary depending on model inputs which are by and large uncertain.

Breach side slope along the breach width and depth is a factor that specifies the shape of breach opening. However accurate predicting the breach side slope angle is of secondary importance to predicting the width and depth (Wahl, 1998). Therefore, this factor is not considered in the MCR breach analysis. As the side slope is insensitive to the breach outflow, this study used a fixed breach side slope angle of 45° (1h:1v slope) in BREACH modeling.

Most empirical breach time equations are for the final breach time (Tf). However, the breach time that is critical in flood hazard analysis is not Tf but the time to peak flow (Tp). Breach time estimated by physical breach models (e.g., BREACH, FLDWAVE, etc.) tends to be longer than those using empirical breach time equations. This long breach time may result in less conservative breach flood hazard.

Physically-based breach models are available to aid in the prediction of breach parameters. There are many reported physical breach models (Wahl, et al., 2008; Gee, 2008, etc.), including NWS-BREACH, SIMBA, HR-BREACH, FIREBIRD-BREACH, etc. Although none are widely used, the most notable is the National Weather Service BREACH models (Fread, 1988: NWS-BREACH or BREACH hereafter). Physical models are intended to simulate the hydraulic and erosion processes associated with flow from overtopping or piping failures of a dam and obtain breach parameters and breach outflow hydrographs.

Primary weakness of these physical breach models is the fact that they do not adequately model the erosion process that dominates the breaching of cohesive-soil embankments (e.g., Hahn et al. 2000). This is why many hydrologists recommend using multiple methods (both

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Wahl (2004) concludes "The results from use of the physically based NWS-BREACH model were reassuring because they fell within the range of the values obtained from the regression-based methods."

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Due to the observation, sensitivity analysis on uncertain parameters was undertaken by PNNL.

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Unsure what the author means. Probably that some physical models were used without calibration.

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Not sure how this is justified. The side slope has great bearing on conveyance area and discharge.

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Unsure which MCR breach analysis the author is referring to here.

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Not sure this is a relevant statement. BREACH produces a hydrograph which will have a peak flow.

empirical equations and physical models) in a dam breach analysis. However, both STP and PNNL used only the selective empirical equations or modeling. Instead, the author tried to use all available empirical equations and multiple models in a comprehensive manner.

6.3.2 Breach Parameter Estimation Using Empirical Equations

The listed below summarizes the empirical regression equations for estimating width and time of breach and peak breach outflow (Wahl, 1997):

- U.S. Bureau of Reclamation (1988)
  - Average breach width (m), $B_{avg} = 3\ h_w$
  - Failure time (hr), $Tf = 0.011\ B_{avg}$
  - Peak flow (m^3/s), $Qp = 19.1\ h_w^{1.85}$

- MacDonald and Langridge-Monopolis (1984)
  - Volume of eroded material (m^3), $B_{avg} = V_{er}/A = 0.0261\ (V_w\ h_w)^{0.769}/A$
  - Failure time (hr), $Tf = 0.0179 V_{er}^{0.364}$ for bounding, or $Tf = 0.0704\ (V_{er})^{0.2874}$ for best fit
  - Peak flow (m^3/s), $Qp = 3.85\ (V_w\ h_w)^{0.411}$ for bounding

- Von Thun and Gillette (1990)
  - Average Breach width, $B_{avg} = 2.5h_w + C_b$
  - Failure time (hr), $Tf = B_{avg}/(4h_w+61)$ for highly erodible dam materials
  - Peak flow (m^3/s), not available

- Froehlich (1995a, b)
  - Average breach width (m), $B_{avg} = 0.1803\ K_o\ V_w^{0.32}\ h_b^{0.19}$
  - Failure time (hr), $Tf = 0.00254\ V_w^{0.53}\ h_b^{-0.9}$
  - Peak flow (m^3/s), $Qp = 0.607\ V_w^{0.295}\ h_w^{1.24}$

where

h_w = depth of water above breach invert (m)
V_w = volume of water stored (m^3)
C_b = offset factor which is a function of reservoir size:
K_o = overtopping multiplier (1.4 for overtopping, 1.0 for piping)
h_b = height of breach (m), and
A = breach cross section area (m^2).

The MLM(1984) paper presents a bound failure time equation but not a best fit failure time equation. Therefore, the author obtained the above best-fitted breach time equation using 14 earth-fill dam failure data in the MLM's paper. The figure below shows the regression analysis where the historical data of erosion volume and time were log-transformed to fit a linear line as in the case of MLM's breach erosion volume equation. The linear regression equation was back-transformed to get an exponential breach time equation as shown above. The MLM best-fitted breach time equation is used in this study for consistency with other equations.

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The author presented a best-fit failure time equation attributed to MacDonald and Langridge-Monopolis (1984) above. Unsure if this is the equation fitted by the author to data presented by MacDonald and Langridge-Monopolis (1984).

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ing

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So HA came up with his own regression. Has this been peer reviewed?

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equation

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Which equation does this refer to? It sounds like a NRC result? Is it the equation on the figure below?

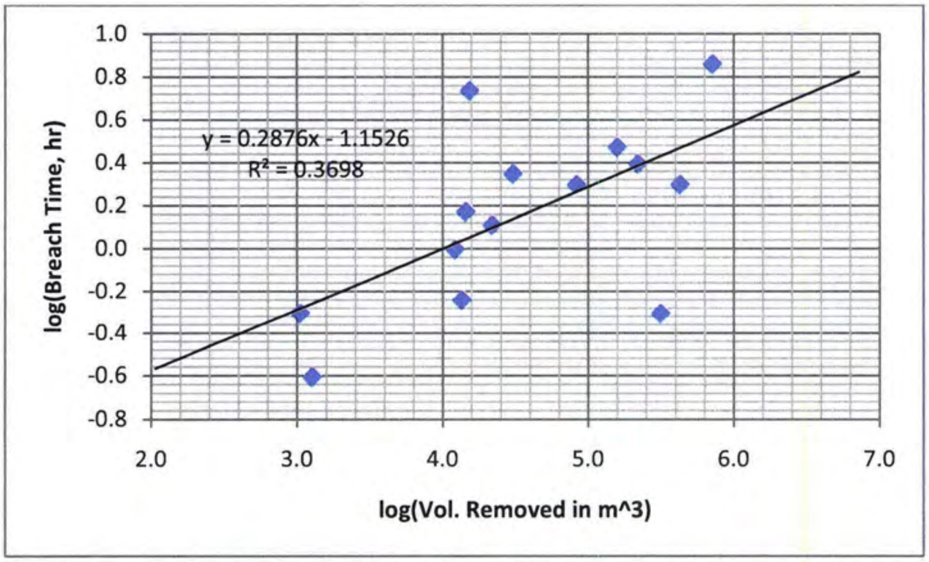

Figure 6.3-2. The best-fitted MLM breach time regression line that was obtained with the dam breach data provided by the MLM paper (1984).

The author used an initial MCR level before breach of 50.9 ft MSL and the bottom ~~levee~~ elevation of 29 ft MSL. The corresponding reservoir head (h_w) and storage volume (V_w) are 21.9 ft and 6.653×10^9 ft^3. Table 6.3-1 lists estimated breach parameters using the empirical equations above. The MLM breach width estimate in Table 6.3-1 is based on the scenario of no scour hole. The Table 6.3-2 compares the MLM breach widths for different scour hole depths. Table 6.6-3 lists peak breach outflow rates using 10 different empirical equations which are presented in Wahl's paper (1997). Some of the peak flow equations in Wahl (1997) were excluded in Table 6.3-2 because they are a bounding equation. However the envelop USBR peak flow equation is included because it is used in Table 6.3-1.

Table 6.3-1. MCR breach parameters estimated by four sets of empirical equations.

Equation	Breach Width (ft)	Failure Time (hr)	Peak Flow (cfs)
Bureau of Reclamation	66	0.66	22,607
MacDonald and Langridge-M	1736	2.54	217,000
Von Thun and Gillette	235	2.68	n.a. (1)
Froehlich	417	7.98	60,078

Note: (1) n.a. = not available.

(2) Cross section area of 5281 ft^2 at the breach bottom level of 29 ft MSL was used for the MLM breach width equation here.

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HA computed the log-log regression.

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embankment

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From Wahl (2004)

"Using the MacDonald and Langridge-Monopolis equation, the estimate of eroded embankment volume and associated breach width for the top-of-joint-use scenario is also comparable to the other equations. However, for the top-of-flood-space scenario, the prediction is much larger than any of the other equations, and in fact is unreasonable because it exceeds the dimensions of the dam."

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Figure 10 in Wahl (1998) shows predicted vs observed breach widths (in meters) using Von Thun & Gillette, Froelich and US BUREC predictions equations. None of the observed or predicted widths for breach case in the DSO database exceed 250 m (820 ft).

Table 6.3-2. MLM breach width estimates using different scour hole scenarios

Scenario	Scour Hole Depth (ft)	Levee Erosion Volume (ft3)	MLLM Breach Width (ft)
Case 1	0 (no scour hole)	5281	1736
Case 2	10 ft	8754	1047
Case 3	20 ft	12309	745

Table 6.3-3. Breach preak flow equations and their estimated values for MCR.

Author	Equation	Qp(cfs)
Kirkpatrick (1977)	$Qp=1.268(h_w+0.3)^{2.5}$	5,754
SCS (1981)	$Qp=16.6(h_w)1^{.85}$	19,648
USBR (1982)-Envelop	$Qp=19.1h_w1^{0.85}$	22,607
Singh & Snorrason-1 (1984)	$Qp=13.4h_d^{1.89}$	46,105
Singh & Snorrason-2 (1984)	$Qp=1.776\ S^{0.47}$	458,501
Hagen (1982)	$Qp=0.54(S.h_d)^{0.5}$	826,102
MLM (1984)	$Qp=1.154(V_w*h_w)^{0.412}$	**217,247**
Costa (1985)	$Qp=0.981*(s*h_d)^{0.42}$	271,919
Evans (1986)	$Qp=0.72*V_w^{0.53}$	578,760
Freohlich (1997)	$Qp=0.607*V_w^{\ 0.295}\ h_w^{\ 1.24}$	60,078
Average of 10 estimates		**251.000**
Average of last 5 estimates		**336,000**

Where, Qp = peak breach outflow in (m^3/s)

S = Reservoir storage (m^3), and

h_d = height of dam (m).

Based on the result of the above estimations, the following ~~discussions were~~ made:

- Both the Bureau of Reclamation and Von Thun and Gillette methods tend to underestimate breach width and peak flow rate because they are estimated as a function of storage or head only.

- USBR's dam breach manual (Wahl, 1998) states that the Froehlich's equation appears to be the best predictor for the breach widths less than 164 ft because the breach width records used to developed for the Froehlich equation is small. In other words, the Froehlich's breach width equation is not applicable to the MCR breach case.

- Therefore, this study considers the MLM method only. The MLML breach width range from 745 ft to 1736 ft with and without scour hole. The MLM estimates are comparable to the results of BREACH and FLDWAV simulation as shown later.

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Figure 13 in Wahl (1998) shows relationship between depth of water at dam and peak outflow (observed and predicted by several methods). The 6.7 m (21.9 ft) depth used by author intersects the cluster of predictions from about 300-1000 m3/s (10,600-35,300 cfs). The highest observed peak flow with this depth of water is about 4000 m3/s (141,260 cfs).

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points can be

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Is this based on statement on page 64 of Wahl (1998): "Froelich's relation appears to be the best predictor for the cases with observed breach widths less than 50 meters."

Note also that Wahl (1998) concludes "Based on this analysis, it appears the Froelich relation is one of the better available methods at this time for direct prediction of of peak breach discharge."

Also, see Tony Wahl's reply to Ken See's e-mail on June 22, 2011.

- Also the MLM peak flow estimate of 217,000 cfs is roughly consistent with an average (251,000) of 10 peak flow estimates shown below. It should be noted that the first five peak flows are underestimated because they ignore the MCR storage volume. Excluding these five, an average of peak flow is 336, 000 cfs.

The above discussions lead to a conclusion that the breach parameters estimated by STP (B=417 ft, Tf=1.7 hrs, Qp=130,000 cfs) are underestimated, resulting in an underestimation of the DBF level. This conclusion will be further validated using the simulation of numerical breach and flow models later. However, ~~the~~ STP' and PNNL's dam breach analyses ignored or omitted the above empirical equations-derived information partly or entirely without proper justification.

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Number: 1 Author: D3K890 Subject: Cross-Out Date: 7/15/2011 12:14:45 PM -04'00'

Number: 2 Author: D3M760 Subject: Highlight Date: 7/21/2011 9:58:56 AM -04'00'
PNNL used physics-based approach.

6.4 Breach Parameter Simulation Using the BREACH Model

The author used the NWS-BREACH model to supplement the breach parameter estimates by empirical equations. The BREACH model is probably the best known physically-based numerical breach model. BREACH model predicts breach parameters (size, shape, time of ~~formulation~~) and breach outflow hydrograph emanating from a breached earth dam (Fread, 1993 & 2000). This model simulates coupling of the specific erosion processes and the flow process by breach. The BREACH model also accounts for the dynamic effects of the flow within the upstream reservoir using a simple mass-balance type reservoir routing and the tailwater effects on the flow through the breach. Because the BREACH model use only a limited immediate reach of the tailwater channel, additional flow routing model is needed for simulating the flood hazard far downstream.

The BREACH model uses the the Meyer-Peter & Muller (MPM) sedimentation equation for dam breach process and the 1-D Manning's uniform flow equation for the breach outflow. These equations are function of the Manning's roughness coefficient which has been known as one of the most critical parameters in the BREACH model (refer to Figure 6.4-1). STP uses n-value of ~~0.005~~ in their BREACH modeling, which seems too small for sediment-oriented breach flows. This n-value of 0.05 results in the peak breach outflow of 83,000 cfs which is also small compared to the estimates by other methods.

PNNL (2011) selected n-value of ~~0.075~~ without justification. PNNL performed a sensitivity analysis, but part of their sensitivity analysis is erroneous due to a faulty BREACH setup. Therefore, the author investigated the proper n-value for the MCR breach, followed by a supplemental sensitivity analysis of the MCR BREACH model.

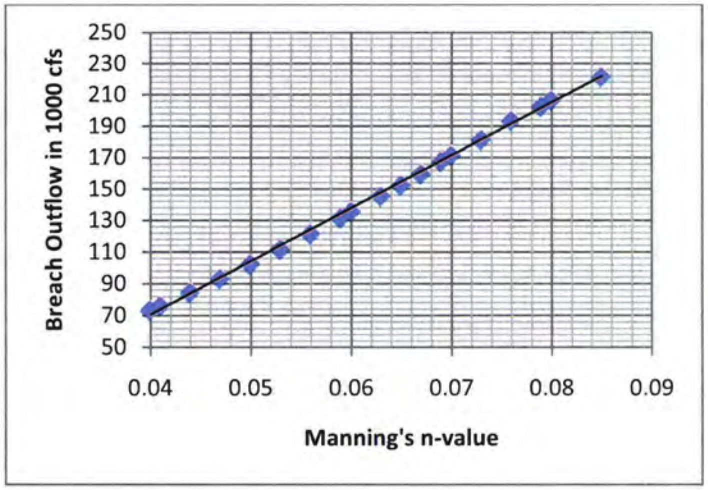

Figure 6.4-1. The relation between Manning's n-value and breach peak outflow obtained from BREACH simulations (without scour hole effects).

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Number: 1 Author: D3M760 Subject: Inserted Text Date: 7/21/2011 9:59:37 AM -04'00'
"formation"

Number: 2 Author: D3K890 Subject: Highlight Date: 7/15/2011 12:17:20 PM -04'00'
Unsure what the author means by this statement.

Number: 3 Author: d3b102 Subject: Replacement Text Date: 7/12/2011 11:15:21 AM -04'00'
0.05

Number: 4 Author: D3M760 Subject: Highlight Date: 7/21/2011 10:02:29 AM -04'00'
Based on sensitivity analyses and consideration of likely upper limit on roughness.

Number: 5 Author: d3b102 Subject: Replacement Text Date: 7/12/2011 11:16:05 AM -04'00'
0.075

Number: 6 Author: D3M760 Subject: Highlight Date: 7/22/2011 5:56:49 PM -04'00'
Is this referring to the tailwater cross-section? Tailwater cross section should be at the d/s end of the breach, not the open "valley" otherwise all dam analyses should have a tailwater cross section the width of the "valley."

6.4.1 Manning's Roughness Coefficients

There are many referenced n-values applicable to channel and overland flows. However, there is no specific guidance that directs to select n-value for dam breach simulations. STP's FSAR states that, for BREACH modeling, an n-value within the range from approximately 0.025 to 0.05 was considered reasonable, while a higher n-value of 0.08 is still considered within the range of feasibility. The n-value of 0.075 is used in SER. The BREACH manual proposed to use the ~~Straler's~~ method to estimate n-value. However, the estimated n-value with this equation is too small because the size of the embankment material (silty clay) is small (0.001 mm).

Chow (~~1969~~) states that n-values for sediment laden flow is higher than without it. Since n-value in the BREACH model is critical in breach process, this subsection ~~estimate~~ the n-value for the MCR breach considering the embankment materials, sand chimney, and cement blocks. The author used ~~the~~ Chow's method to estimate the n-value for the MCR breach. The estimated value was compared with other literature values.

<u>Chow's Method to Estimate n-value</u>

This report estimated Manning's n-value using the method presented by Chow (1959; Arcement and Schneider, 1984). Table 5-5 in Chow (1959) provides a procedure for estimating specific n-value considering the effects of bed materials and shape of channel as:

$$n=(n_b+n_1+n_2+n_3+n_4)m \qquad \text{(Equ. 6.4-1)}$$

where

- n_b is the base n value
- n_1 is the irregularly factor
- n_2 is the channel cross section factor
- n_3 is the obstruction factor
- n_4 is the vegetation factor, and
- m is the meandering factor.

The parameters in equation (6.4-1) are estimated based on the above two references as following:

- n_b is 0.02 for earth bed materials which consisted of sand with the median diameter of 0.4 mm.
- n_1 is 0.015 for an average of moderate and severe irregular channels caused by badly eroded or sloughed sides of canals. The maximum n_1 for severe irregularity is 0.02.
- n_2 ranges from 0.01 to 0.015 for frequently alternating of channel cross section due to the constriction at the breach invert area and wide expansion at the breach outlet. The rapid contraction and expansion create highly turbulent breach flow.

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Number: 1 Author: D3K890 Subject: Highlight Date: 7/15/2011 12:22:33 PM -04'00'

Not sure what the author means here. Maning's n is a parameter used to represent roughness of channel bed and sides. This parameter is known to vary with bed material, flow depth, bed condition, presence of gravel, boulders, and vegetation, and sinuosity of the channel.

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Strickler's

Number: 3 Author: D3K890 Subject: Inserted Text Date: 7/15/2011 12:25:41 PM -04'00'

1959

Number: 4 Author: d3b102 Subject: Replacement Text Date: 7/12/2011 11:20:06 AM -04'00'

estimates

Number: 5 Author: D3K890 Subject: Cross-Out Date: 7/15/2011 12:24:59 PM -04'00'

- n_3 ranges from 0.02 to 0.03 for appreciable obstruction caused by the littering of clay cement blocks that fall and cover about 40% of the breach surface area during the breach process.
- n_4 is 0.005 for small vegetation. The maximum n_4 is 0.01.
- m is 1 for minor (minimum) meandering.

Using the above, the low and high n-values are computed as 0.07 and 0.085, respectively, with an average n-value of 0.0775. These estimated values are compared with other tabulated and referenced n-values as the table below.

Table 6.4-1. Comparison of the Manning's roughness coefficients (n-values) for difference sources.

Source	n-value		
	Minimum	Medium	Maximum
FSAR	0.025	0.05 (used)	0.08
SER	0.025	0.05	0.075 (used)
Chow Method (this study)	0.07	0.0775	0.085
Table 5-6 in Chow (1959) – major stream (B>100 ft) with irregular and rough section	0.035	-	0.10
FLO-2D MANUAL – overland flow	-	0.07	-
Estimated based on the MLM peak flow of 217,000 cfs	-	0.085	-

In summary, n-values for the breach process could range from 0.025 to ~~1.0~~. The estimated n-value in this study for the MCR breach considering the breach condition, sand chimney, and cement blocks is 0.0775, but the BREACH model in this report uses an n-value of 0.075 in line with the SER.

6.4.2 BREACH Sensitivity Analysis

The BREACH model uses the following input parameters:

- Dam materials: grain size, porosity, unit weight, n-value, frictional angle, cohesive strength, and D90/D30 ratio for both inner and outer materials

- Dam properties: (1) for clay materials, PI index, critical shear stress, and critical stress coefficient; (2) slope of upstream and downstream faces, and slope of core; (3) for grass cover, length, condition, max erosion velocity, max sediment concentration

- Breach parameters: ratio of initial width to depth, width of dam, tailwater slope, particle size (D50), ration of D90 and D30, max bottom width, and max top width

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Number: 1 Author: D3K890 Subject: Highlight Date: 7/15/2011 12:46:53 PM -04'00'

The author does not give details, but it seems like this is how Chow's method (1959), which is meant for drainage channels and natural streams was applied by the author:

1. "low" estimate of Manning's n appears to use nb=0.02, n1=0.015, n2=0.01, n3=0.02, and n4=0.005. With m set to 1.0, the estimated value of n is 0.07.

2. "high" estimate of Manning's n appears to use nb=0.02, n1=0.02, n2=0.015, n3=0.03, and n4=0.01. With m set to 1.0, the estimated value of n should be 0.095. Unsure if the author has a typographical error for the "high" value he reported, or used a different set of constituent values in the equation.

Number: 2 Author: D3K890 Subject: Highlight Date: 7/15/2011 1:41:17 PM -04'00'

The author's estimate of Manning's n value is 0.07 to 0.085 (0.095?). Here the author seems to say that the acceptable range could vary from 0.025 to 0.1.

Fread (1988) presented example applications of BREACH for failure analyses of the a) Montaro Dam using a Manning's n of 0.02 and b) Spirit Lake using a Manning's n of 0.018.

Fread (1988) also presented example applications of BREACH for failure analysis of the Teton Dam and suggested that because D50 of 0.03 mm for the dam was too fine, Strickler's equation was judged not applicable for estimation of Manning's n. Instead, Fread (1988) estimated a Manning's n value of 0.013 from a Darcy friction factor based on D50 and Moody's diagram.

Chris Meader of BOSS International, one of the distributors of BREACH, states regarding Manning's n: "BREACH requires that you define the Manning's roughness value for the cross section downstream of the reservoir, and the Manning's roughness value of the earthen material that makes up the dam. The cross section roughness simply represents the on-site field conditions that the flow will encounter after it leaves the failed reservoir structure. This should be the same as what you enter into DAMBRK. The dam material roughness represents the roughness of bare earth, which can be quite small since earthen material (clay, etc.) tends to be quite smooth."

Number: 3 Author: d3b102 Subject: Replacement Text Date: 7/12/2011 11:20:27 AM -04'00'

0.1

- Elevation: initial reservoir level, top and bottom elevations of dam, initial piping elevation, and spillway crest.

- Others: tailwater cross section, simulation time step, and output control.

PNNL performed a BREACH sensitivity analysis to determine the sensitivity of inputs on the MCR levee breach process and outflows (PNNL, 2011: refer to the PNNL's calpack). Based on the sensitivity analysis, PNNL concluded that:

- The BREACH model predictions are very sensitive to n-value.

- The BREACH model predictions are slightly sensitive to internal frictional angle.

- The breach predictions for peak flow and time and breach width are fairly insensitive to initial piping elevation, cohesive strength, frictional angle,

- The model predictions are very sensitive to n-value.

However, PNNL made erroneous sensitivity conclusions for (1) length of levee as not at all sensitive; and (2) tailwater cross section as slightly sensitive. Therefore, the author performed an additional sensitivity analysis for n-value, levee length, tailwater cross section, and scour hole as described below.

Formulate a Base Run Scenario

To start with, the author postulated the following four MCR breach scenarios for which the result of BREACH simulations is summarized in the table below:

- FSAR scenario: {N=0.05,C=300 lb/ft2, phi=20^{O}}

- SER scenario: {N=0.075,C=200 lb/ft2, phi=15^{O}}, but all other parameters are the same as those for the FSAR scenario.

- RUN1: A corrected SER scenario with tailwater cross section and max levee length of 3000 ft. Note: MCR BREACH gives a runtime error if width is greater than 3000 ft.

- RUN2: RUN 1 with a 10-ft scour hole.

Table 6.4-2. Comparison of BREACH simulations with different MCR breach scenarios..

Run	Condition	Peak				Final Breach[(2)]			
		Qp	Tp	Breach Width		Qf	Tf	Breach Width	
		kcfs	hour	Top, ft	Bot., ft	kcfs	hour	Top, ft	Bot.,ft
FSAR	n=0.05	83	6.25	412	361	26	22.51	498	446
SER	n=0.076	128	1.99	574	463	23	20.51	747	637
RUN1	Corrected	194	3.29	989	878	24	17.39	1259	1149
RUN2	Scour hole	269	2.12	703	562	26	21.64	861	720

Notes:

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Number: 1 Author: d3b102 Subject: Comment on Text Date: 7/12/2011 11:21:52 AM -04'00'
repeated statement???

Number: 2 Author: D3M760 Subject: Highlight Date: 7/22/2011 6:04:22 PM -04'00'
This produced a smaller breach width, not bigger. This produced a smaller breach width, not bigger, as suggested in Section 4 p.7, 4th bullet.

- Kcfs ~~is the~~ 1000 cubic feet per second.
- RUN 2 is the base run for the ~~following~~ sensitivity analysis.

The tables above and below provide peak breach parameters as well as those at the end of the breach process in order to compare those using empirical equations. The final breach time is almost infinite, but they were determined using the criteria of (1) the difference between the headwater level and tailwater level (e.g., HY-TWD parameter from BREACH output) become less than 1 ft or (2) breach outflow is less than 27,000 cfs which is about 10% of RUN 2 peak outflow. The peak parameter values for the "SER run" are from the PNNL calpack – These are slightly different from the simulation of this study. From this simulation, the following ~~discussions~~ are made:

- The result of this simulation indicates that the corrected BREACH model produces higher peak outflow, and even higher if the effects of scour hole are considered.

- The breach time simulated by the BREACH model tends to be longer than the ones by empirical equations. However, the tail of the outflow is not significant in determining the DBF level, thus ignored.

- The breach process continues after the peak flow, but the rate of the breach process is relatively slow compared to that before reaching the peak outflow.

<u>BREACH Sensitivity Runs</u>

Table 6.4-3. Sensitivity of Manning's n-value to BREACH model results.

Run ID	n-value	Peak				Final Breach			
		Qp	Tp	Breach Width		Qf	Tf	Breach Width	
		kcfs	hour	Top, ft	Bot., ft	Kcfs	hour	Top, ft	Bot.,ft
R11	0.10	323	1.06	820	679	60	9.68	1018	877
R12	0.08	280	1.52	723	583	46	12.24	892	751
BASE	0.075	269	2.12	703	562	26	21.64	861	720
R13	0.06	224	2.96	623	483	31	16.5	745	604
R14	0.04	139	6.20	481	340	32	19.4	557	416

Table 6.4-4. Sensitivity of scour hole size to BREACH model results.

Run ID	Hole Depth	Peak				Final Breach			
		Qp	Tp	Breach Width		Qf	Tf	Breach Width	
	Ft	kcfs	hour	Top, ft	Bot., ft	kcfs	hour	Top, ft	Bot.,ft
R21	0	195	3.63	1029	918	26	16.40	1676	1565
R22	5	252	3.04	875	749	26	18.93	1107	981
BASE	10	269	2.12	703	562	26	21.64	861	720
R23	15	271	1.80	595	439	26	24.65	716	560
R24	20	267	1.64	518	347	26	14.8	620	450

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denotes

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Number: 3 Author: D3K890 Subject: Inserted Text Date: 7/15/2011 1:46:50 PM -04'00'
following

Number: 4 Author: D3K890 Subject: Highlight Date: 7/15/2011 1:49:21 PM -04'00'
Perhaps the author means "breach parameters at peak discharge."

Number: 5 Author: D3M760 Subject: Inserted Text Date: 7/21/2011 10:14:23 AM -04'00'
"points"

Number: 6 Author: D3M760 Subject: Highlight Date: 7/21/2011 10:16:59 AM -04'00'
I don't think his handling of the tailwater cross section is correct. See comment above.

Number: 7 Author: D3M760 Subject: Highlight Date: 7/21/2011 12:17:15 PM -04'00'
This corresponds to Run 2, not Run 1. Isn't Run 1 the base?

Number: 8 Author: D3M760 Subject: Highlight Date: 7/21/2011 12:53:58 PM -04'00'
It is not clear how HA includes the hole in BREACH. Is it included as part of the tailwater cross-section? Or was some other method used?

Notes: R21 and RUN1 in Table 6.4-2 are identical.

Table 6.4-5. Sensitivity of tailwater cross section to BREACH model results.

Run ID	Tailwater Width	Peak				Final Breach			
		Qp	Tp	Breach Width		Qf	Tf	Breach Width	
	Ft	kcfs	hour	Top, ft	Bot., ft	kcfs	hour	Top, ft	Bot.,ft
BASE	3000	269	2.12	703	562	26	21.64	861	720
R31	2000	244	0.77	611	470	24	24.42	849	709
R32	1500	219	0.78	570	430	24	24.92	798	658
R33	1200	204	0.98	570	429	25	33.39	781	641
R34	1000	195	0.79	526	386	26	25.14	771	630
R35	800	159	1.10	489	349	23	29.33	679	540
R36	600	122	0.80	388	248	25	29.84	567	426

Note: (1) On these sensitivity runs, levee length was fixed to 3000 ft.

(2) The tailwater section width greater than 3000 ft ~~give~~ runtime error on the MCR BREACH simulation.

Table 6.4-6. Sensitivity of levee length to BREACH model results.

Run ID	Dyke Length	Peak				Final Breach			
		Qp	Tp	Breach Width		Qf	Tf	Breach Width	
	Ft	kcfs	hour	Top, ft	Bot., ft	kcfs	hour	Top, ft	Bot.,ft
BASE	3000	269	2.12	703	562	26	21.64	861	720
R461	600	265	0.62	600	488	26	15.39	600	488
R462	500	220	0.47	500	388	25	16.74	500	388
R47	400	167	0.39	400	288	25	22.04	400	288

Note: On these sensitivity runs, width of tailwater cross section was fixed to 3000 ft.

The above four tables summarize the result of sensitivity analyses for n-value, scour hole depth, width of the bottom tailwater section, and levee length, respectively. From this sensitivity runs, the following ~~discussions~~ are made:

- The BREACH model predictions are very sensitive to n-value. This was also noted in the PNNL's calpack and Figure 6.4-1.
- The BREACH model predictions are very sensitive to the scour hole depth. ~~Ehe~~ effective of scour hole is nonlinear - The peak outflow rates increase with increasing hole depth. The flow rates are peaked (271 kcfs) at the hole depth of 15 feet, then decrease thereafter. It seems that this peak flow is the probable maximum MCR breach outflow, if scour hole is considered in the breach process.
- Both peak time and breach width is inversely proportional to the hole depth. The deeper the scour hole is, the more the initial breach outflows is. However, scour hole reduces breach width. In conclusion, considering the effects of breach scour hole induce high peak breach outflows and more severe flood hazard than without considering it.

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Number: 1 Author: D3K890 Subject: Highlight Date: 7/22/2011 6:11:04 PM -04'00'

The numbers for Run1 in Table 6.4-2 are not identical to numbers for Run21 in Table 6.4-4.

Number: 2 Author: d3b102 Subject: Replacement Text Date: 7/12/2011 11:27:12 AM -04'00'

gives

Number: 3 Author: D3M760 Subject: Highlight Date: 7/22/2011 6:16:29 PM -04'00'

This analysis does not address sensitivity, considering that values below computed breach width completely erode the dam. Also, intermediate values that might show a response are not included in the analysis.

Number: 4 Author: D3M760 Subject: Inserted Text Date: 7/21/2011 10:19:27 AM -04'00'

"points"

Number: 5 Author: d3b102 Subject: Replacement Text Date: 7/12/2011 11:28:34 AM -04'00'

The

- In contrast to the PNNL's conclusions, the sensitivity analysis by this study demonstrates that BREACH predictions are very sensitive to both [1] levee length and the width of tailwater cross section. These are not variable. However, use of correct and realistic values for these two inputs is needed in BREACH modeling.

Number: 1 Author: D3M760 Subject: Highlight Date: 7/21/2011 10:26:53 AM -04'00'

This was NOT demonstrated. See comments to the table above.

6.5 Breach Outflow Simulation Using the FLDWAV Model

The FLDWAV model (Fread, 1993) is a generalized unsteady-flow model for open channels. It replaces the DAMBRK, combining its dam breach simulation capabilities and providing new hydraulic simulation procedures. FLDWAV can simulate dam failures caused by either overtopping or piping. The program can also represent the failure of two or more dams located sequentially on a river. The program is based on the complete equations for unsteady open-channel flow (St. Venant equations). This model has options to simulate various external and internal boundary conditions and hydrologic structures.

Fread (2000) states that "BREACH should be used judiciously and with caution - BREACH model is intended to be an auxiliary method for determining the breach parameters and should be used in conjunction with statistical and range of magnitude data from historical breaches." Therefore, the BREACH model has not been directly incorporated into FLDWAV. However, FLDWAV can easily simulate breach outflow hydrographs using breach parameters (width and time) as input. This feature gives an advantage of bypassing the burden of identifying the BREACH input parameters, especially the Manning's n-value. After setting up the FLDWAV model for the MCR breach case, the author performed a comprehensive sensitivity analysis to understand the behavior of the MCR FLDWAV model.

<u>FLDWAV Model Set-up</u>

The MCR FLDWAV model was set up with available data from FSAR and RAI responses. The MCR FLDWAV model consists ~~with~~ three hydrologic components: MCR reservoir routing, MCR ~~levee~~ reach, and tailwater routing. All of these hydrologic components are simulated in a 1-~~dimentional~~ way because FLDWAV does not have capability to simulate 2-D flows. The MCR FLDWAV model uses five channel cross sections: The first cross section is for MCR reservoir with levee, and remaining four are for tailwater reaches.

First, MCR reservoir is used as an upstream boundary condition. Relevant reservoir input to FLDWAV include MCR stage-surface area rating curve, starting and bottom reservoir elevations, and upstream inflow (discharge) boundary condition. There is no actual upstream inflow into the MCR during the breach, but small inflow values were input to the model to provide ~~a~~ stability ~~of the~~ channel routing - FLDWAV gives a runtime error without this fictional input.

Second, ~~levee~~ breach input include elevations of top and bottom of ~~levee~~, weir coefficient, breach time, breach width, initiating reservoir level, reservoir bottom level, levee crest length, initial piping level, and breach side slope (1h:1v). These values are same as those for the MCR BREACH model. Again, width and time of breach is the most critical parameters in determining breach outflows.

Third, tailwater flow condition was assumed as a 1-D flow on a wide channel (4 miles width) to mimic the 2-D overland flow condition at the power block area. This tailwater condition is more realistic than that of the MCR BREACH model that uses only one tailwater cross section with a

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Number: 1 Author: D3M760 Subject: Highlight Date: 7/21/2011 10:29:38 AM -04'00'

Need reference on which this statement was based. Have the developers of FLDWAV said this?

Number: 2 Author: d3b102 Subject: Comment on Text Date: 7/15/2011 3:29:45 PM -04'00'

PNNL's understanding is that FLDWAV has no predictive capability for estimation of breach parameters; rather the breach formation parameters need to be prescribed to FLDWAV. BREACH could be used as a means to determine reasonable ranges for the breach parameters. Site-specific information (geotechnical reports) and general literature were available for configuring BREACH so bypassing the burden of identifying the BREACH input parameter was considered less realistic than incorporating site-specific and general information.

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of

Number: 4 Author: D3K890 Subject: Inserted Text Date: 7/15/2011 3:30:45 PM -04'00'

embankment

Number: 5 Author: D3M760 Subject: Inserted Text Date: 7/21/2011 10:30:11 AM -04'00'

"dimensional"

Number: 6 Author: D3K890 Subject: Cross-Out Date: 7/15/2011 3:32:12 PM -04'00'

Number: 7 Author: D3K890 Subject: Cross-Out Date: 7/15/2011 3:32:29 PM -04'00'

Number: 8 Author: D3K890 Subject: Inserted Text Date: 7/15/2011 3:32:22 PM -04'00'

to

Number: 9 Author: D3K890 Subject: Inserted Text Date: 7/15/2011 3:32:53 PM -04'00'

MCR embankment

Number: 10 Author: D3K890 Subject: Inserted Text Date: 7/15/2011 3:33:05 PM -04'00'

the embankment

Number: 11 Author: D3K890 Subject: Highlight Date: 7/15/2011 3:33:33 PM -04'00'

Number: 12 Author: D3M760 Subject: Highlight Date: 7/22/2011 6:23:55 PM -04'00'

See comments above regarding tailwater cross section above. Additional comment, even if the there is a river channel into which the dam normally discharges, it cannot be assumed that the channel d/s of the dam is the tailwater channel. Because a piping failure may occur anywhere along the dam. The point is that if Fread intended the tailwater cross section to be "infinitely" wide, then why include it in the input?

width of 3000 ft. Tailwater input include number of cross sections, Manning's n-value for channel (specified from 0.03 to ~0.08, but actual n-values are calculated by the model internally depending on the condition and depth of flows), length and slope of each channel reach, and cross section geometry.

After setting up the MCR FLDWAV model, it was tested for the STP's breach scenario (e.g., B=415 ft, Tf=1.7 hours). The test run provide the same peak outflow (128,000 cfs), validating that the model was set up correctly. The author performed a comprehensive sensitivity analysis in order to understand the behavior of the MCR FLDWAV model.

FLDWAV Sensitivity Runs

The author simulated the MCR FLDWAV model with different widths and times of breach. The upper bound of the breach parameters are determined conservatively (5000 ft width and 3 hours breach time). For discrete values of breach parameters, 42 FLDWAV runs were performed. The following two tables list simulated peak outflows and times to peak. They were also plotted in the following two figures.

Tp\B_avg	500 ft	750 ft	1000 ft	1500 ft	2000 ft	3000 ft	5000 ft
0.5 hr	285	415	539	735	830	933	1077
1.0 hr	277	391	466	535	590	670	779
1.5 hr	265	331	369	425	469	534	635
2.0 hr	241	280	309	357	398	454	523
2.5 hr	210	244	271	312	348	397	475
3.0 hr	189	219	241	280	306	361	426

Table 6.5-1. Peak breach outflows (in 1000 cfs) simulated by the MCR FLDWAV model with different breach widths and times.

Tp\B_avg	500 ft	750 ft	1000 ft	1500 ft	2000 ft	3000 ft	5000 ft
0.5 hr	0.5	0.5	0.5	0.5	0.5	0.425	0.35
1.0 hr	1.0	1.0	1.0	0.9	0.8	0.7	0.6
1.5 hr	1.5	1.5	1.35	1.20	1.05	0.9	0.75
2.0 hr	2.0	1.8	1.6	1.4	1.3	1.1	0.9
2.5 hr	2.375	2.125	1.875	1.625	1.5	1.375	1.125
3.0 hr	2.7	2.4	2.1	1.95	1.65	1.5	1.2

Table 6.5-2. Peak breach time (in hour) simulated by the MCR FLDWAV model with different breach widths and times.

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Number: 1 Author: d3b102 Subject: Comment on Text Date: 7/12/2011 11:42:25 AM -04'00'

Does FLDWAV report what these values are computed to be?

Number: 2 Author: D3K890 Subject: Highlight Date: 7/15/2011 3:35:11 PM -04'00'

417 ft?

Number: 3 Author: D3K890 Subject: Cross-Out Date: 7/15/2011 3:35:30 PM -04'00'

Number: 4 Author: d3b102 Subject: Comment on Text Date: 7/12/2011 11:49:39 AM -04'00'

Not clear of the purpose of red shading? It is explained later but maybe should be noted in caption.

FLDWAV peak breach time (as input) is superceded by internal computation when input is unrealistic.

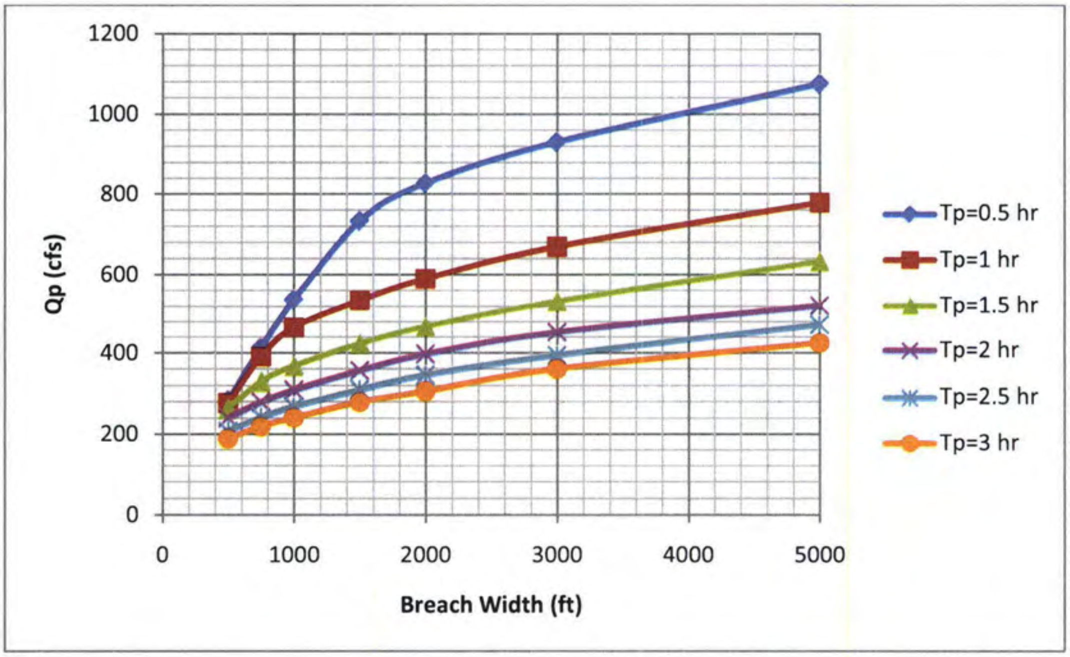

Figure 6.5-1. Breach peak outflows estimated by FLDWAV with different breach widths and times.

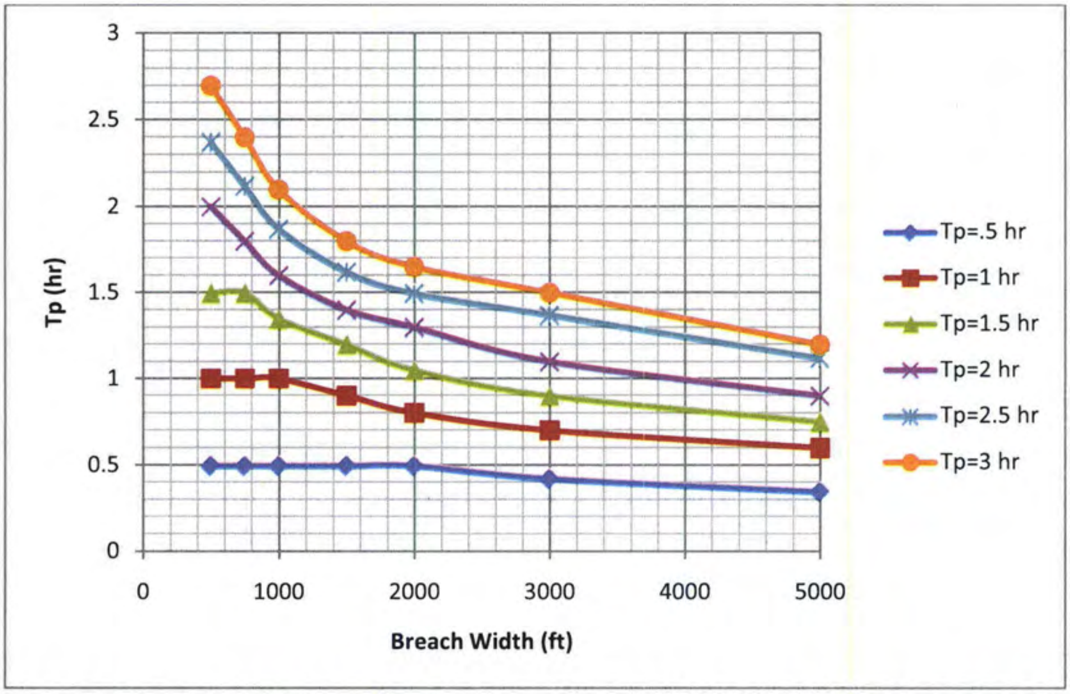

Figure 6.5-2. Breach peak outflows estimated by FLDWAV with different breach widths and times.

This page contains no comments

From these sensitivity runs, the following discussions were made:

- The peak outflow is proportional to breach width, but the rate of increase is nonlinear. The peak outflow increases ~~shapely~~ [1]r the breach width less than 2000 ft, but the rate of increase is reduced thereafter.

- The pink color cells in Table 6.5-2 are the case [3]here FLDWAV[2]-simulated breach peak time is less than the corresponding peak time input. The reduction is more significant for wider breach widths. The reduction indicates that peak time in FLDWAV[4] model input is unrealistically small so that the FLDWAV[5] model adjusts the outflow and time to peak based on the breach condition and geometry (breach width and others).

- The two figures above are useful in determining the peak outflow and time given the breach width and time obtained from empirical equation. In cases where peak outflow and time are estimated independently, it is necessary to filter and adjust the parameters using the FLDWAV runs. Figure 6.5-1 and 6.5-2 are useful in adjusting the peak outflow and time as shown in Subsection 6.6.6.

Page: 35

Number: 1 Author: d3b102 Subject: Replacement Text Date: 7/12/2011 11:45:33 AM -04'00'
sharply

Number: 2 Author: D3K890 Subject: Cross-Out Date: 7/15/2011 3:40:05 PM -04'00'

Number: 3 Author: D3K890 Subject: Inserted Text Date: 7/15/2011 3:40:36 PM -04'00'
s

Number: 4 Author: D3K890 Subject: Cross-Out Date: 7/15/2011 3:40:09 PM -04'00'

Number: 5 Author: D3K890 Subject: Cross-Out Date: 7/15/2011 3:40:13 PM -04'00'

6.6 Flood Routing Using the FLO-2D Model

In general, dam breach flood analysis in a river valley or channel has been performed with 1-dimensional (1-D) flood routing models such as HEC-RAS or FLDWAV. In such cases, dam breach flood wave is confined laterally by natural abutments and well-defined downstream river channels, therefore an implicit assumption of velocity vectors normal to the river cross-sections is reasonable for dam breach analysis. However, levee breach flood routing for a large off-stream reservoir constructed on flat ground must be done with a 2-dimentional (2-D) simulation with velocities and flows dispersing in any direction. The MCR levee breach flood at the power block area should be handled by a 2-D hydrodynamic model. The main difference between 1-D and 2-D simulations is that, channel cross sections for 1-D simulation are replaced with a continuous topographic surface for 2-D simulation.

The MCR levee breach flood analysis in this study is based on a combined approach of 1-D simulation of breach outflows and 2-D routing of overland flow after the breach. Because the RMA2 model that was used by STP is not available to the NRC staff, the author used the FLO-2D model. FLO-2D is a dynamic routing model that simulates channel flow, unconfined overland flow and street flow. It can simulate a flood over complex topography and roughness while reporting on volume conservation which is the key to accurate flood routing. FLO-2D uses a full dynamic wave momentum equation and a central finite difference routing scheme with eight potential flow directions over a system of square grid elements. The equations of motion in the FLO-2D finite difference model grid are defined as a quasi two-dimensional model, which are solved by computing the average flow velocity across a grid boundary one direction at a time. FLO-2D does not distinguish between subcritical and supercritical flow because the momentum equation used in the model has no restrictions when computing the transition between the flow regimes. Because of this advantageous feature, the FLO-2D model has been widely used in simulating levee breach floods.

6.6.1 FLO-2D Model Set-up

The author set up the MCR FLO-2D model to simulate the MCR breach induced flood at the power block area. The FLO-2D model has options to simulate various types of flows, including overland flow, channel flow, flood plain, levee, dam and levee breach, hydraulic structures, ~~street~~ flow, rain-runoff-infiltration process, sediment transport, mud and debris flow, and others. However, the MCR FLO-2D model utilizes only the overland flow option and limited options for levee and levee breach because it uses the breach outflow hydrograph simulated by the FLDWAV model as upstream boundary inflow. All other options were omitted, making the MCR FLO-2D model very simple and effective in running. The input to the MCR FLO-2D model are as following:

- Static data: surface elevation and n-value in space, drainage features (location, width, depth), features of ponds and reservoirs (extend, elevations, levee information), features of structures and roads (location, elevation, etc).

- Boundary conditions: upstream boundary inflow hydrograph, location of outflow boundary nodes – they are not sensitive to the DBF level.

- ~~Levee~~ information: Location, top and bottom elevation

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Number: 1 Author: d3b102 Subject: Comment on Text Date: 7/15/2011 3:50:17 PM -04'00'

I don't know why RMA2 would not be available to the NRC. RMA2 is packaged with TABS and SMS and available for download from AQUAVEO. SMS can be installed and is not fully functional without a valid license. The executable for RMA2 is part of that install and its standalone use is not managed under the SMS license. PNNL does not recall any inquiries from the NRC regarding the availability of the RMA2 executable. (need to check zip files to see if executable were delivered with site files). In order to apply RMA2 efficiently (visualize results, modify computational meshes, etc.), a fully licensed version of SMS is needed. Costs for SMS are comparable to that of FLO-2D. The SMS software tool is readily available.

Number: 2 Author: D3M760 Subject: Highlight Date: 7/21/2011 1:57:20 PM -04'00'

RMA2 is the same. I have run it with Froude numbers greater than 1.

Number: 3 Author: d3b102 Subject: Comment on Text Date: 7/12/2011 11:52:27 AM -04'00'

This is one place that references would be useful.

Number: 4 Author: D3M760 Subject: Highlight Date: 7/22/2011 6:28:36 PM -04'00'

Does FLO-2D actually simulate breaches?

Number: 5 Author: D3K890 Subject: Highlight Date: 7/15/2011 3:52:39 PM -04'00'

sheet?

Number: 6 Author: D3M760 Subject: Inserted Text Date: 7/21/2011 1:59:04 PM -04'00'

"sheet"?

Number: 7 Author: D3M760 Subject: Highlight Date: 7/21/2011 2:12:59 PM -04'00'

Are these from BREACH or FLDWAV? Reported scenarios in the table below do not include peak breach flows smaller than 189 kcfs.

Number: 8 Author: D3M760 Subject: Inserted Text Date: 7/21/2011 2:01:57 PM -04'00'

Embankment

- Breach information: location (nodal points) and width of breach
- Numerical tolerance input: surface detention factor, allowable percent change in flow depth, and dynamic wave stability factor.

The boundary of the MCR FLO-2D model was determined by a trial and error so that the model area is small enough to run efficiently but is large enough to avoid the effects of the downstream boundary on the estimated DBF level (Figure 6.6-1). The model domain in this study is larger than that of the STP's RMA2 model. The model domain is divided into uniform, square grid nodes (200 ft resolution). It has a total of 29948 nodes. The nodal points for structures and MCR ~~levee~~ were assigned in the model.

Breach failure location was identified as the ~~closed levee section~~ from the proposed units 3 and 4. The model was set up so that the location of the breach section is used as an inflow boundary. This input boundary feature ~~enable to exclude~~ the entire MCR area, which ~~reduce~~ model run time significantly. The MCR FLO-2D model uses the breach outflow hydrograph simulated by the FLDWAV model as input as was done in the STP's RMA2 model.

At the site, the typical n-value of 0.045 was assigned. The author performed a sensitivity analysis of the MCR FLO-2D model with changing n-values from 0.025 to 0.08. The result shows that the n-value is not sensitive to the DBF levels – The change of DBF levels is only 0.36 ft which is substantially smaller than the change by breach parameters. This is because FLO-2D has a flow depth variable n-value function so that the n-values are adjusted by the model internally rather than using the input n-values.

Other model parameters for MCR, ~~levee~~, and structures are the same as those used in the STP's RMA2 model. The run time of the MCR FLO-2D model is quite extensive – Simulation of a 20-hour MCR breach flood took more than 10 hours. Most of the cases, the MCR FLO-2D model simulations were stopped externally by the author after 5 hours in model time because the maximum flood levels at the site occurs 2 to 3 hours.

This study, like the STP's analysis, used a sunny day ~~levee~~ breach which is caused by seepage through ~~levee~~ or foundation, or slide of ~~levee~~ materials. That is, MCR antecedent rain or storm surge effects on the MCR breach flood were not considered in the MCR FLO-2D model. Evaporation and recharge losses of the MCR breach water were also ignored in flood routing as they are insignificant in determining the DBF level.

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Number: 1 Author: d3b102 Subject: Comment on Text Date: 7/12/2011 12:18:53 PM -04'00'
What kind of downstream boundary condition was used?

Number: 2 Author: D3K890 Subject: Inserted Text Date: 7/15/2011 3:54:31 PM -04'00'
embankment

Number: 3 Author: D3K890 Subject: Inserted Text Date: 7/15/2011 3:55:15 PM -04'00'
closest embankment section

Number: 4 Author: d3b102 Subject: Replacement Text Date: 7/12/2011 11:58:30 AM -04'00'
enables the exclusion of

Number: 5 Author: d3b102 Subject: Replacement Text Date: 7/12/2011 11:58:46 AM -04'00'
reduces

Number: 6 Author: d3b102 Subject: Comment on Text Date: 7/12/2011 12:00:14 PM -04'00'
Does FLO-2D report the value of the internally computed n-values?

Number: 7 Author: D3K890 Subject: Inserted Text Date: 7/15/2011 3:58:17 PM -04'00'
embankment

Number: 8 Author: D3K890 Subject: Inserted Text Date: 7/15/2011 3:58:31 PM -04'00'
s

Number: 9 Author: D3K890 Subject: Inserted Text Date: 7/15/2011 3:59:00 PM -04'00'
between

Number: 10 Author: D3K890 Subject: Inserted Text Date: 7/15/2011 3:59:19 PM -04'00'
embankment

Number: 11 Author: D3K890 Subject: Inserted Text Date: 7/15/2011 3:59:36 PM -04'00'
the embankment

Number: 12 Author: D3K890 Subject: Inserted Text Date: 7/15/2011 3:59:51 PM -04'00'
embankment

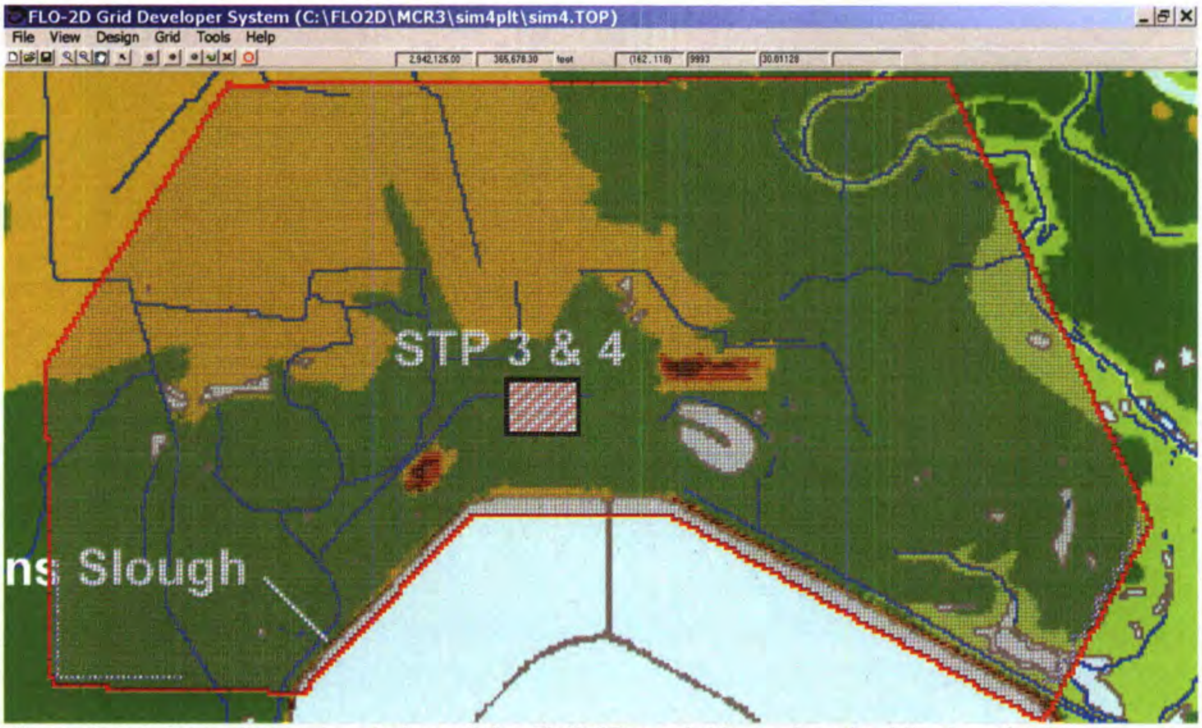

Figure 6.6-1. Two-dimensional view of the FLO-2D model boundary (red line) and grid.

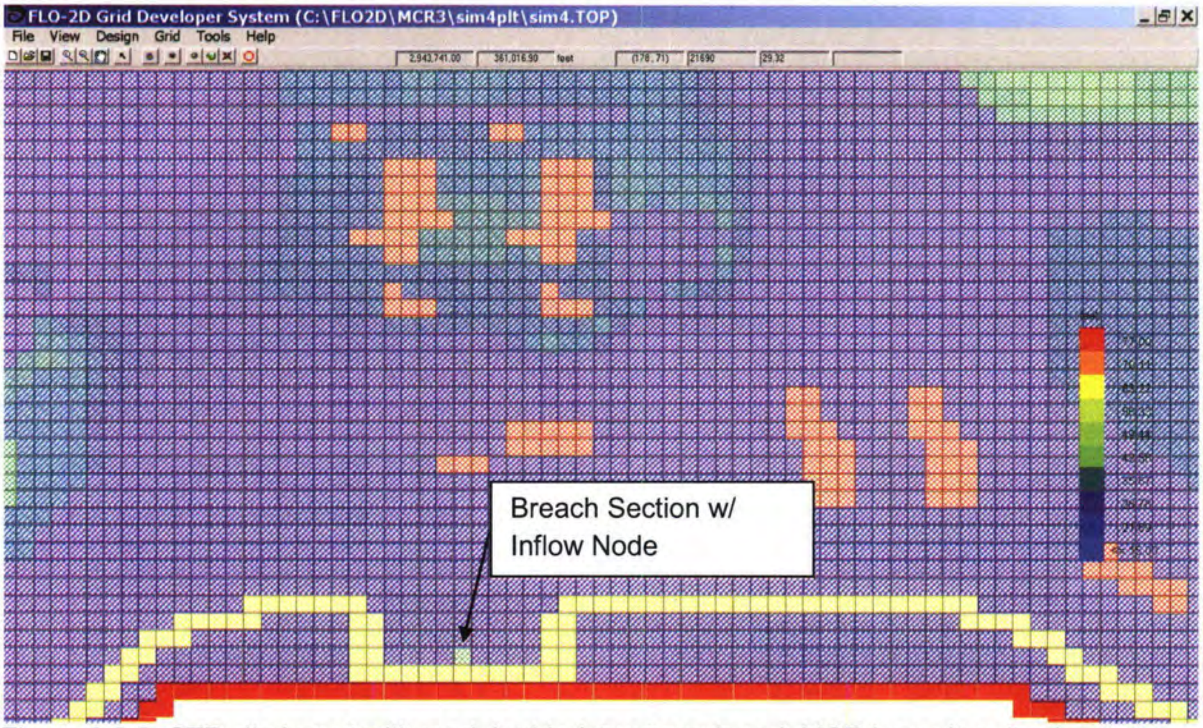

Figure 6.6-2. STP site layout with model grids (structures in red, MCR levee in yellow).

This page contains no comments

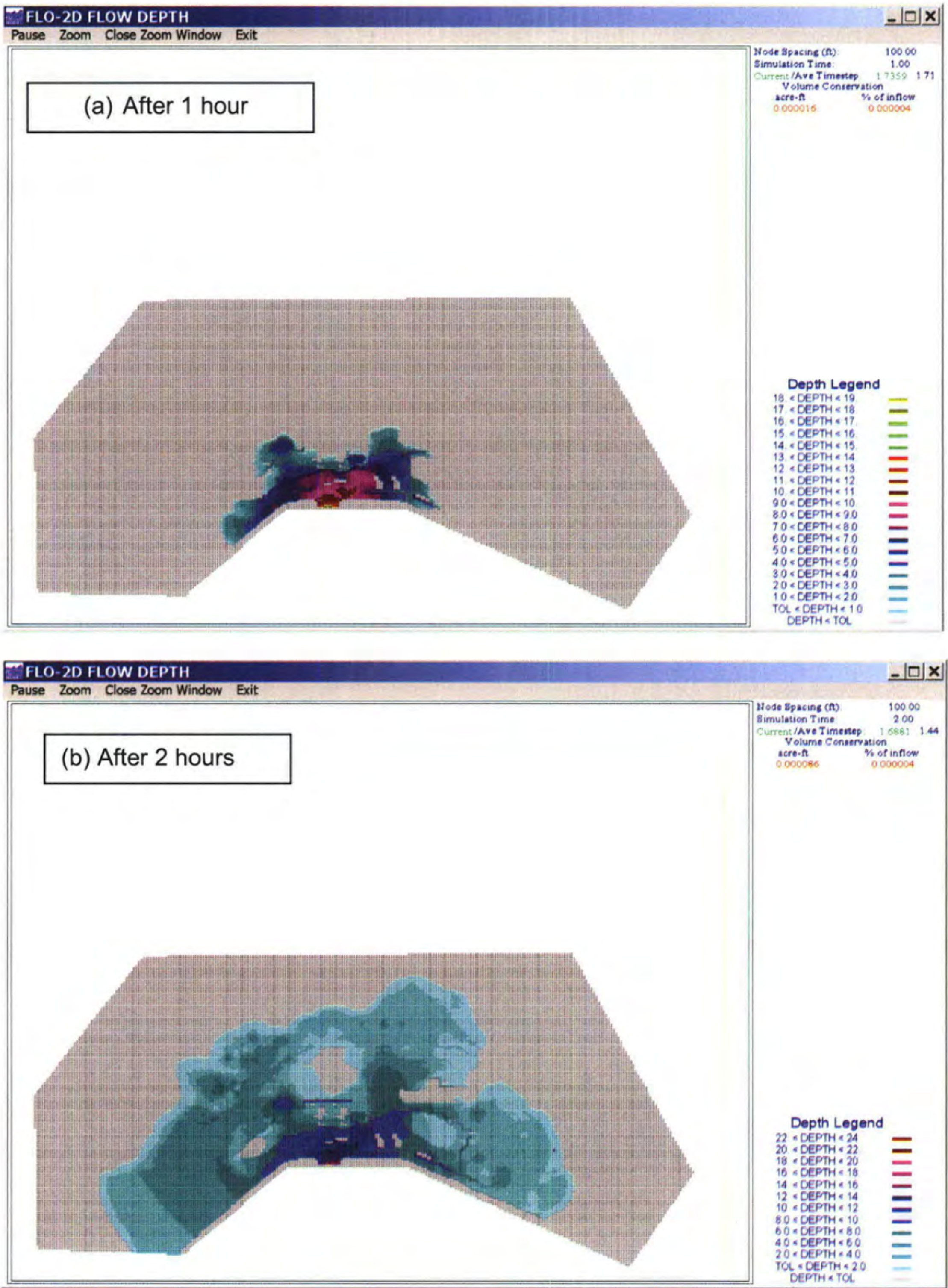

Figure 6.6-3. An example of breach flood propagations simulated by MCR FLO-2D at (a) 1 hours after initiating a breach (b) after 2 hours, and (c) after 3 hours.

T Number: 1 Author: d3b102 Subject: Comment on Text Date: 7/12/2011 12:11:07 PM -04'00'

Depth in ft (assumption)? So a depth of 6ft in area of interest would correspond to a 40 ft msl? Am I reading this correctly?

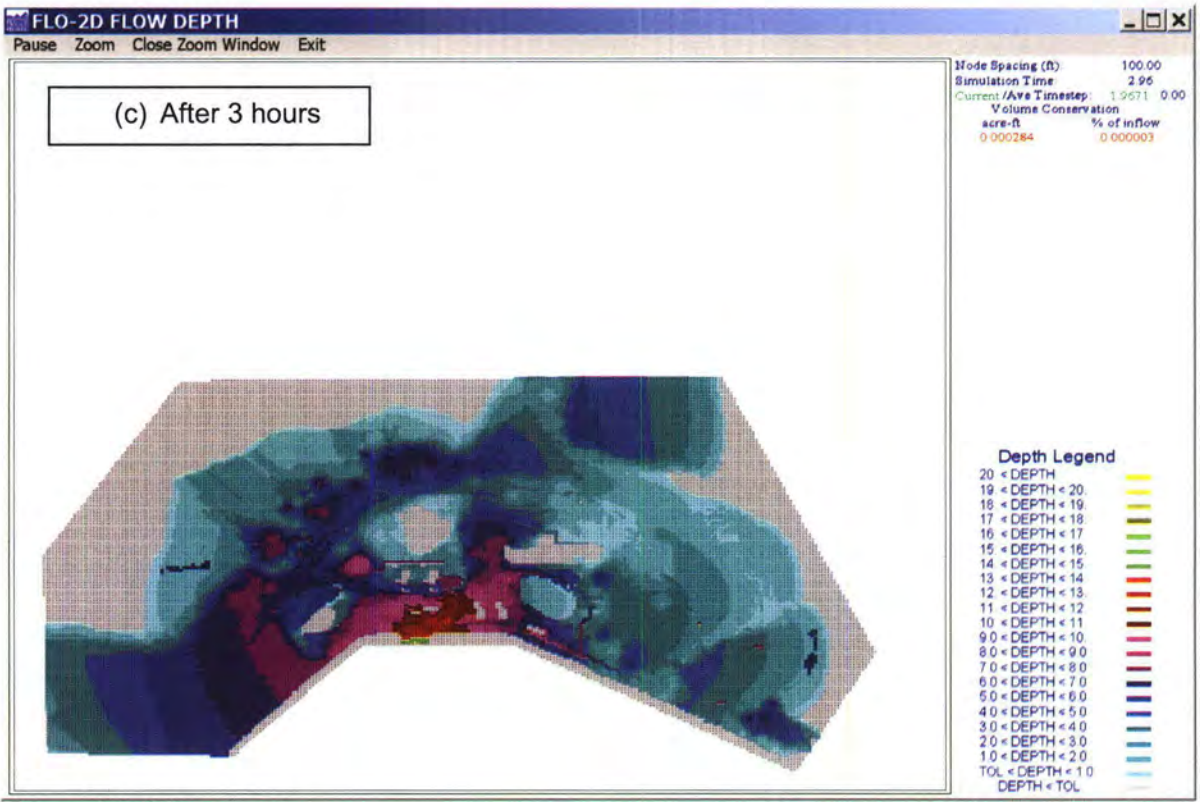

Figure 6.6-3 (continue)

This report includes some simulation maps to help understand the behavior of the MCR FLO-2D model. These maps are snapped from the FLO-2D pre/post processing software during the simulation.

Figure 6.6-3 shows a typical MCR breach flood propagation in space simulated by the MCR FLO-2D, where the propagation is displayed in terms of the maximum flow depths. This example is for the simulation of the STP's MCR breach scenario case where breach width is 417 ft and breach time is 1.7 hours. As shown from this figure, ~~levee~~ breach water flows towards the north ~~with~~ inundating the site, divides into the East and West directions almost equally, and then eventually flows to south to either the Colorado River or the Little Robin Slough.

Figure 6.6-4 displays the land surface elevation map used in the MCR FLO-2D model. The land surface elevation data were provided by STP, which are digitized to the MCR FLO-2D model grid cells using the ~~fLO-~~2D pre-processing software. Some of the cell elevations were adjusted based on the configuration of the drainage system, MCR reservoir-levee system, and proposed plant facilities to incorporate control of flow directions.

Figure 6.6-5 and 6.6-6 display simulated maximum flood elevation and maximum flood velocity maps, respectively, using the MCR FLO-2D model. The simulation condition of these maps is the same as that of Figure 6.6-3 (STP's breach scenario).

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Number: 1	Author: D3K890	Subject: Inserted Text	Date: 7/15/2011 4:02:52 PM -04'00'

MCR embankment

Number: 2	Author: d3b102	Subject: Cross-Out	Date: 7/12/2011 12:12:42 PM -04'00'

Number: 3	Author: D3K890	Subject: Inserted Text	Date: 7/15/2011 4:03:31 PM -04'00'

i

Number: 4	Author: d3b102	Subject: Comment on Text	Date: 7/12/2011 12:21:27 PM -04'00'

Not sure what is meant here. Does this mean that adjustments were made to reflect the existances of topographical features that were not resolved due to the resolution of the model?

Number: 5	Author: d3b102	Subject: Replacement Text	Date: 7/12/2011 12:13:29 PM -04'00'

FLO

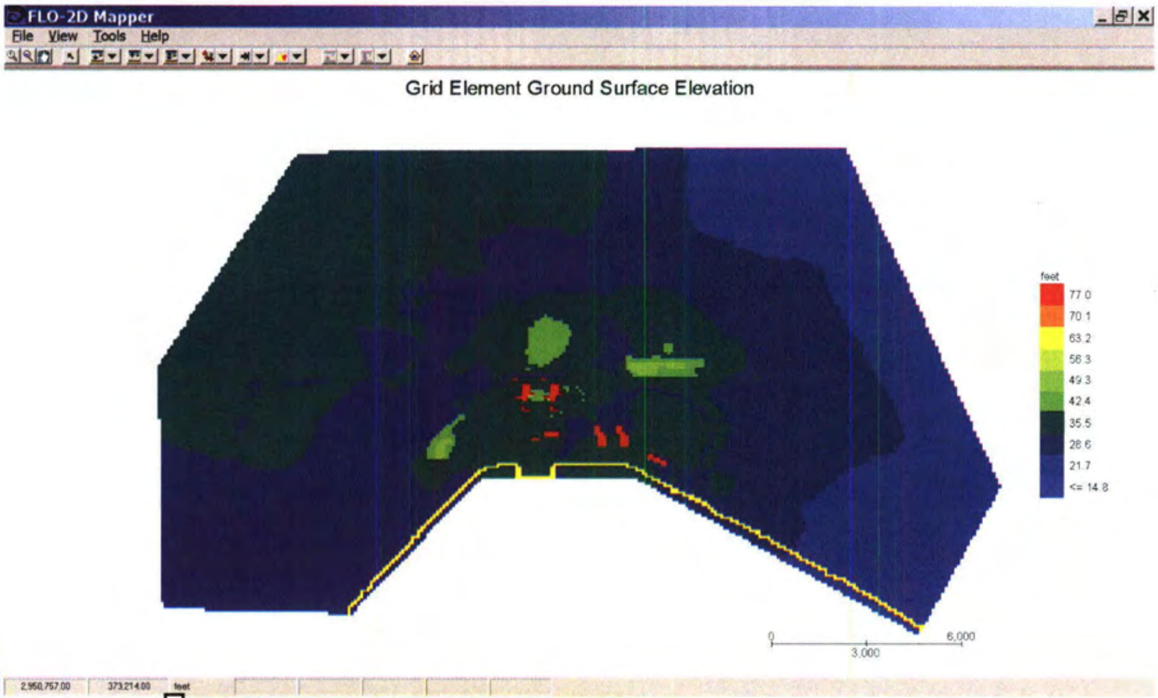

Figure 6.6-4. MCR FLO-2D surface elevation map.

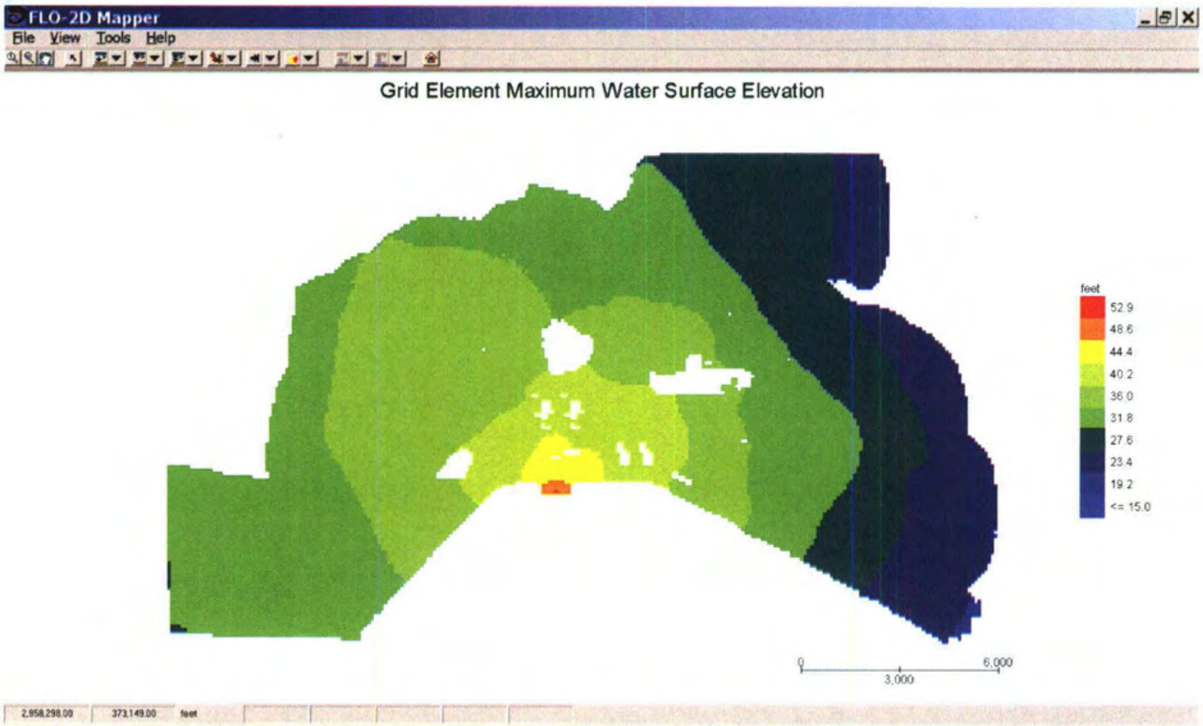

Figure 6.6-5.An example of the FLO-2D simulated maximum water surface elevation map.

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Number: 1 Author: d3b102 Subject: Comment on Text Date: 7/12/2011 12:23:10 PM -04'00'

The annotation in this figure indicated a FE rather than a FD numerical approach. Is MSL implied for vertical datum?

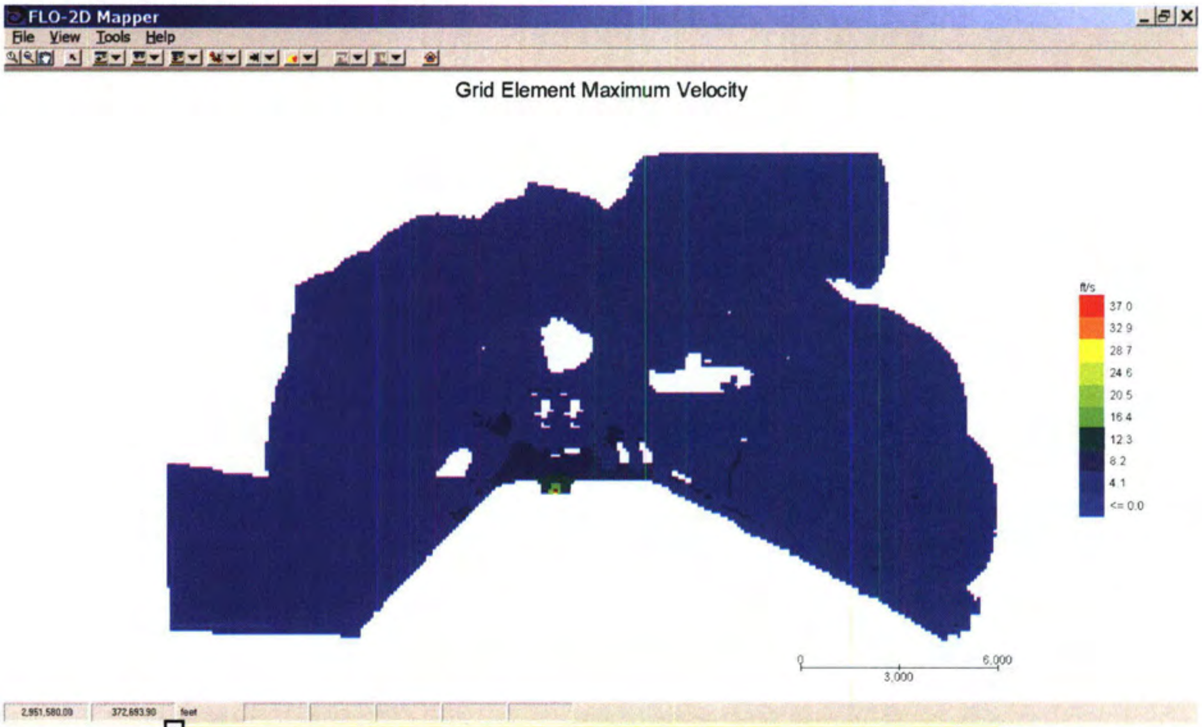

Figure 6.6-6. An example of the FLO-2D simulated maximum flood velocity map.

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Number: 1 Author: d3b102 Subject: Comment on Text Date: 7/12/2011 12:24:43 PM -04'00'
Don't see color variation in area of interest.

6.6.2 FLO-2D Model Simulation

As in the case of FLDWAV sensitivity runs, the author performed a simulation of MCR FLO-2D model for different breach widths and breach peak times. The purpose of this simulation is to set up the relation of breach scenario (characterized by breach width and peak time) and the resulting DBF level. The ranges of breach widths and peak times were determined based on the breach parameters estimated using empirical breach equations. The breach parameters were discretized (3 breach widths and 6 breach times), making a total of 18 runs. In order to monitor the simulated peak flood levels around the Units 3&4 power block area, the following 6 model nodes (node numbers) were chosen as:

17297: South of the Unit 4 Reactor Building
17553: South of the Unit 4 Service Building
18319: South of the Unit 4 UHS
17306: South of the Unit 3 Reactor Building
17562: South of the Unit 3 Service Building
18328: South of the Unit 3 UHS

On each run, the peak flood level during the simulation period (usually 5 hours) at each selected node was read from the FLO-2D output file. These peak flood level values are tabulated ~~on the table~~ below, and the maximum flood level (so called the DBF level) was determined from the peak flood levels at 6 nodal points. In all runs, cell number 18328 (south of the Units 3 UHS) gives the highest flood level.

Table 6.6-1. The result of simulated peak flood levels simulated by the MCR FLO-2D with different breach peak outflows and times.

Run ID	Breach width	Tp-Input	Qp	Tp	Peak Flood Level for different Cell ID 17297	17553	18319	17306	17562	18328	Max flood level
	ft	hr	Kcfs	hr	ft	ft	ft	ft	ft	ft	ft
b500t05	500	0.5	285	0.5	42.78	43.27	44.11	43.74	43.57	44.17	44.17
b500t10	500	1.0	277	1	42.63	43.14	43.92	43.60	43.47	44.03	44.03
b500t15	500	1.5	265	1.5	42.50	42.98	43.72	43.43	43.29	43.84	43.84
b500t20	500	2.0	241	2	42.03	42.53	43.26	42.97	42.83	43.34	43.34
b500t23	500	2.5	210	2.375	41.48	41.96	42.62	42.39	42.23	42.74	42.74
b500t27	500	3.0	189	2.7	41.01	41.47	42.11	41.89	41.74	42.19	42.19
b500t05	750	0.5	415	0.5	44.82	45.40	46.36	45.90	45.75	46.46	46.46
b500t10	750	1.0	391	1	44.62	45.12	46.06	45.62	45.47	46.19	46.19
b500t15	750	1.5	331	1.5	43.70	44.24	45.07	44.72	44.57	45.24	45.24
b500t20	750	2.0	280	1.8	42.85	43.37	44.17	43.83	43.68	44.29	44.29
b500t23	750	2.5	244	2.125	42.19	42.7	43.45	43.15	43.02	43.56	43.56
b500t27	750	3.0	219	2.4	41.67	42.16	42.87	42.60	42.44	42.93	42.93
b500t05	1000	0.5	539	0.5	46.52	47.30	48.28	47.70	47.56	48.36	48.36
b500t10	1000	1.0	466	1	45.64	46.25	47.31	46.87	46.68	47.47	47.47
b500t15	1000	1.5	369	1.35	44.35	44.88	45.80	45.38	45.29	45.95	45.95

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Number: 1 Author: D3K890 Subject: Cross-Out Date: 7/15/2011 4:08:57 PM -04'00'

b500t20	1000	2.0	309	1.6	43.36	43.89	44.70	44.37	44.21	44.85	44.85
b500t23	1000	2.5	271	1.875	42.74	43.23	44.01	43.71	43.54	44.15	44.15
b500t27	1000	3.0	241	2.1	42.09	42.59	43.32	43.02	42.89	43.42	43.42

6.6.3 Relation between DBF Level and Breach Parameters

Using the resulting DBF levels on the table above, the author performed a multiple linear regression analysis to obtain a regression equation to predict DBF level. The independent variable of the regression is DBF level at the Units 3 and 4 area and the dependent variables are peak breach outflow and peak time. The purpose of this regression analysis is to predict DBF level easily without re-simulating the MCR FLO-2D model that requires a substantial run-time. The resulting regression equation is as following:

$$H = 39.26042+0.01757*Qp/1000-0.03168*Tp, \text{ with } R^2=0.9895 \qquad \text{(Equ. 6.6-1)}$$

where

H is the DBF level at the Units 3&4 area in ft MSL
Qp is the peak flow in cfs, and
Tp is the peak time in hour.

The regression to predict DBF level is quite good as the regression statistics shown (refer to Table 6.6-2 and Figure 6.6-7). Qp is more sensitive to the DBF level than Tp for the possible ranges of these breach parameters. To estimate DBF levels, this equation needs peak breach outflow rate and peak time. These parameters could be estimated from empirical equations or by simulating BREACH, FLDWAV, HEC-RAS, or other breach models.

Table 6.6-2. Regression statics for predicting DBF level using peak outflow and time.

Regression Statistics					
Multiple R	0.994732				
R Square	0.989492				
Adjusted R Square	0.988091				
Standard Error	0.183661				
Observations	18				
ANOVA					
	df	*SS*	*MS*	*F*	*Significance F*
Regression	2	47.64701	23.8235	706.2698	1.44968E-15
Residual	15	0.505972	0.033731		
Total	17	48.15298			

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Number: 1 Author: d3b102 Subject: Comment on Text Date: 7/12/2011 12:34:31 PM -04'00'

I believe "dependent" and "independent" are reversed here. The R2 is high and so appears that H is not sensitive to breach width.

Number: 2 Author: d3b102 Subject: Comment on Text Date: 7/12/2011 12:35:57 PM -04'00'

How does compare to STP results based on FLDWAV & RMA2?

Number: 3 Author: D3M760 Subject: Highlight Date: 7/21/2011 2:20:02 PM -04'00'

It's not clear how this statement was arrived at.

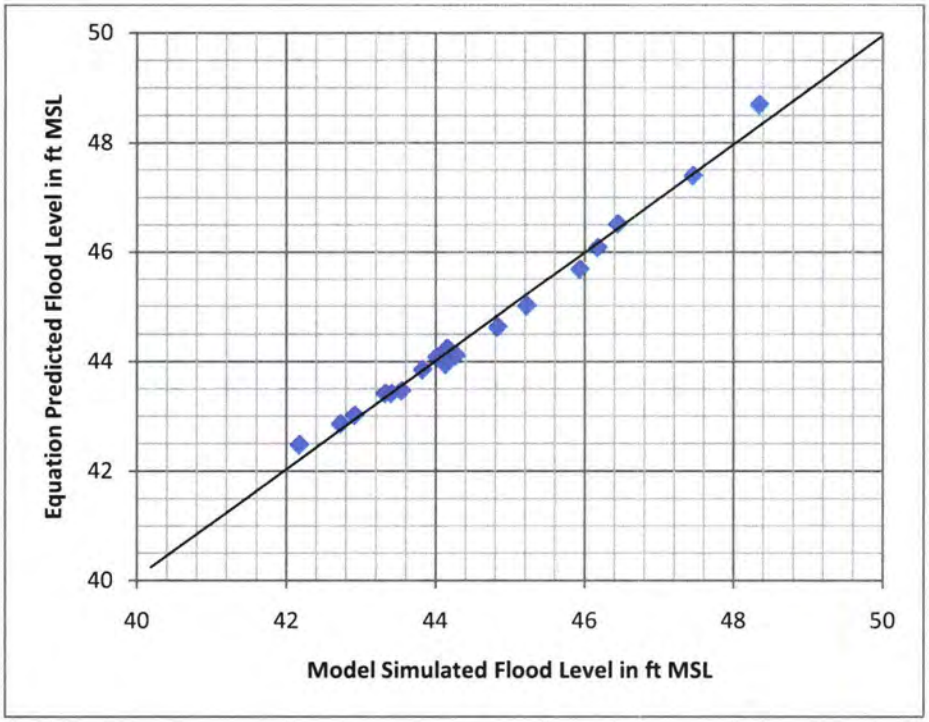

Figure 6.6-7. Linear relationship between the FLO-2D simulated DBF level and predicted one by a multiple linear regression equation.

6.6.4 Validation of FLO-2D Simulations

Direct validation of the MCR FLO-2D model is not possible because there is no actual MCR breach data. However, the model can be validated indirectly using the result of different flood routing models. For MCR breach flood routing, STP used the Delft3D model in the FSAR Revision 0 and the RMA2 model in the FSAR revision 2. The author compared the DBF level obtained by the FLO-2D simulation with those two simulations. Two MCR breach scenarios are used in the comparison: scenario for (B=417 ft, Tf=1.7 hr), and scenario for (B=4757ft, Tf=2.44 ~~hs~~)

STP obtained the maximum flood level of 38.9 ft MSL using the RMA2 model, then determined the DBF level of 40 ft MSL considering a margin for wind wave setup. The author simulated the FLO-2D model with the STP's breach scenario, but obtained the maximum flood level of about 40 ft MSL (without wind setup) which is about 1 foot higher than RMA2 result. The author tried to reduce the FLO-2D DBF level estimates to that of RMA2 by changing n-value, land surface elevations, structure configuration, and breach location and nodes in the model. However, the effort was fruitless nor finding any error in the MCR FLO-2D model. It is author's opinion that STP's RMA2 model could be wrong because they used the unexplained sump zone around the proposed power block area. However it is impossible for the author to check ~~the~~ STP's RMA2

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Number: 1 Author: d3b102 Subject: Comment on Text Date: 8/1/2011 1:22:02 PM -04'00'

NRC submitted RAI related to the configuration and use of Delft3d. Applicant discontinued it use prior to resolution of the RAIs. While the application of Delft3d may ultimately have been sound, PNNL does not feel that future analyses should be based on the results that were never fully reviewed.

Number: 2 Author: d3b102 Subject: Replacement Text Date: 7/12/2011 12:37:05 PM -04'00'

hr

Number: 3 Author: D3M760 Subject: Highlight Date: 7/21/2011 2:23:21 PM -04'00'

Not clear how this was ascertained.

Number: 4 Author: d3b102 Subject: Comment on Text Date: 8/1/2011 1:20:36 PM -04'00'

The artificial sump was explained by applicant in audit and RAI responses. It's effect was also examined by PNNL sensitivity analysis. The impact was determined to be a few tenths of a foot.

Number: 5 Author: D3K890 Subject: Cross-Out Date: 7/15/2011 4:18:34 PM -04'00'

model because the RMA2 model is not available to the staff. Instead of further efforts to refine the MCR FLO-2D model, the author attributed the one feet difference as a modeling uncertainty.

STP provided in the FSAR Revision 0 a table listing the DBF levels estimated by the Delft3D model for different MCR breach widths - They are plotted in Figure 6.6-8. Delft3D-FLOW is a multidimensional hydrodynamic and transport numerical model. This model can simulate unsteady flow and transport phenomena that result from tidal and meteorological forcing on a rectilinear or a curvilinear boundary-fitted grid system. The model solves the Navier-Stokes equations for incompressible fluid using the shallow water and the Boussinesq assumptions. The result of the Delft3D simulation for the STP breach scenario matches that by the RMA2 simulation even though the breach time for the Delft3D simulation is unknown. For the breach width of 4757 ft, FLO-2D and Delft3D gives the DBF levels of 47 ft MSL and 47.7 ft MSL, respectively.

Based on the above comparison results, it was concluded that within the model uncertainty Of 1 foot the MCR FLO-2D provides reasonable DBF level estimates for the range of possible breach parameters.

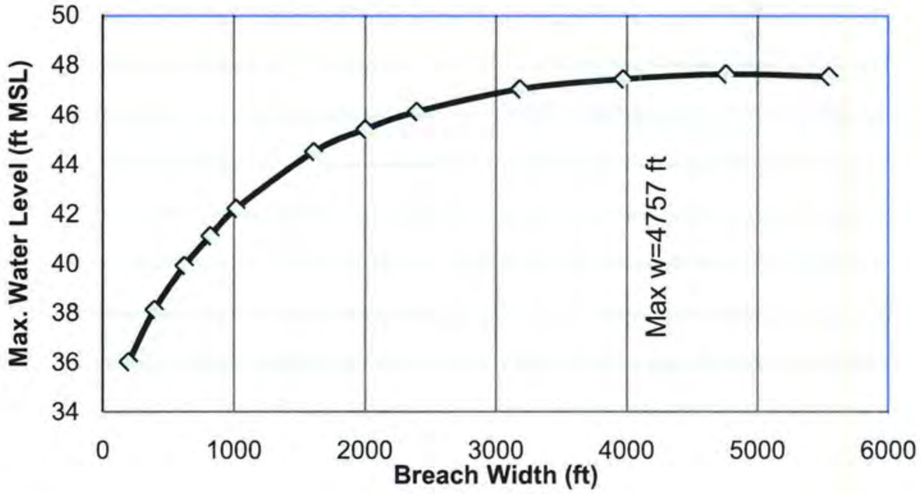

Figure 6.6-8. The relation between MCR breach width and the DBF level estimated by the STP using the Delft3D model (Based on the STP FSAR Revion 0 data).

6.6.5 Sensitivity Analysis of FLO-2D Simulation

The author performed a comprehensive sensitivity analysis for the MCR FLO-2D model. The objectives of this sensitivity analysis were to determine the effects of input variable on the DBF levels and to assure the confidence for the inputs and parameters used in FLO-2D modeling. The following input parameters were used in the sensitivity analysis:

- Model outflow boundary

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I believe it is readily available.

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an

- Grid size
- Manning's n-value
- Land surface elevations, especially near the site.
- Elevations for drainage ditches, Little Robin Slough, and FM521 Highway
- Location and number of the model inflow (breach outflow) nodes
- Existing and proposed structures, and
- Others

This report does not present a detailed result of these sensitivity runs because most of them are not sensitive to the DBF level compared to those for the breach width and time - The input and output files are available in ~~the~~ electronic format. The following ~~is~~ the major findings from this sensitivity test:

- Most of the cases, the model is not sensitive to the above parameters. For instance, the sensitivity analysis of Manning's n-values range from 0.02 to 0.08 (0.04 is used in the model) shows that the change of the corresponding DBF levels is only 0.34 feet which is substantially smaller than that of breach parameters. This is because the FLO-2D model uses corrected n-values based on the flow conditions and flow depths in each cells.

- The sensitivity runs for drainage ditch elevation show that they are not sensitive on the DBF levels even though they affecting the flood level at the beginning of the flood. This is because the capacities of ditches or local streams are too small to control the DBF floods.

- Also, the sensitivity results reveals that the DBL levels simulated by the FLO-2D ~~is~~ not sensitive to the change of the elevations near the plant area, exact locations of facilities, or breach outflow nodal conditions.

6.6.6 Estimation of DBF Levels for Different Scenarios

The author postulated 8 conservative MCR breach scenarios based on the sensitivity analyses using the BREACH, FLDWAV, and FLO-2D models. The descriptions of each scenario are followed:

- MLM-B-D10: Use the MLM breach width equation with **10 ft** scour hole depth; use the MLM breach time equation; estimate Qp and Tp from 6.5-1 and 6.5-2, respectively; n-value used.

- MLM-B-D20: Use the MLM breach width equation with **20 ft** scour hole; use the MLM breach time equation; estimate Qp and Tp from 6.5-1 and 6.5-2, respectively; n-value used.

- MLM Qp: Use the MLM Qp equation; use the MLM Tf for Tp, no scour hole, no n-value.

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Number: 3 Author: D3M760 Subject: Highlight Date: 7/21/2011 2:29:45 PM -04'00'
Wonder how FLO-2D would handle the breach?
I believe there may be depth corrected methods in the newer versions of RMA2 (not included in SMS). One expects that a deep depth, Manning's 'n' to have less influence on momentum. This could be the case within the breach as well.

Number: 4 Author: D3M760 Subject: Highlight Date: 7/21/2011 2:34:09 PM -04'00'
This is surprising. Flow should be diverted around higher bed elevations. Is there significant runup? Or maybe the elevation delta is small. It is not clear from the text how the sensitivity test was made.

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- Avg Qp: Same as above but use an average Qp of 251 kcfs.

- RUN1: Corrected SER scenario with tailwater cross section and max levee length of 3000 ft. Note: MCR BREACH gives a runtime error if width is greater than 3000 ft.

- RUN2: RUN 1 with a 10-ft scour hole.

- RUN23: RUN 1 with a 15ft scour hole.

- RUN24: RUN 1 with a 20-ft scour hole.

Once Qp and Tp were estimated from either empirical equations or breach models, DBF levels are computed using Equation 6.6-1. The table below lists the breach parameters as well as estimated DBF levels for each scenario. The result of this estimation indicates that the reasonably conservative DBF levels at the proposed Units 3 & 4 is about 45 ft MSL considering a margin for wind wave setup.

The DBF level estimates in this table are based on the following three different approaches:

- MPM breach width equation, FLDWAV, and FlO-2D
- MLM peak flow, FLO-2D
- BREACH, FLO-2D

For the BREACH model, n-value of 0.075 is used. The DBF level estimates for the first four scenarios ~~were~~ not ~~relied~~ on the Manning's n-value, but use different empirical equations and FLDWAV. All these approaches produced very consistent DBF estimates.

Table 6.6-3. DBF levels estimated by Equation 6.6-1 with various MCR breach scenarios.

Run ID	B (ft)	Tf (hr)	Qp (kcfs)	Tp (hr)	DBF level H (ft MSL)	Remark
MLM-B-D10	1047	2.54	309	1.87	44.63	Qp &Tp from FLDWAV,10 ft hole
MLM-B-D20	745	2.54	280	2.12	44.11	Qp &Tp from FLDWAV, 20 ft hole
MLM Qp	-	-	217	2.54	42.99	Qp &Tp from FLDWAV, 0 ft hole
Avg Qp	-	-	251	2.54	43.59	Qp &Tp from FLDWAV, 0 ft hole
RUN1	934	-	194	3.29	42.56	Qp &Tp from BREACH, 0 ft hole
RUN2	633	-	269	2.12	43.92	Qp &Tp from BREACH, 10 ft hole
RUN23	516	-	271	1.8	43.96	Qp &Tp from BREACH, 15 ft hole
RUN24	433	-	267	1.64	43.90	Qp &Tp from BREACH, 20 ft hole

6.6.7 Very Extreme MCR Breach Scenarios

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See earlier note regarding location of the tailwater cross section.

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How are the scour holes included in the analyses? There is no explanation.

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a question of judgement

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The breach parameter estimates in Subsection 6.3 are based on the best-fitted linear regression line shown (dot lines in Figure 6.6-9). The problem in using the best-fitted line is that about a half of historical breach records would exceed the best-fit estimate – ~~Th~~ [1]s expected value ~~result~~ [2] in an underestimation. If an extreme conservatism is of concern, one may use a bounding (dashed envelop) line equations. ~~On~~ [3] this regard, the following three extreme scenarios were postulated:

- MLM-B-D0: Use MLM breach width equation with no scour hole, then use FLDWAV to estimate Qp and Tp. It will ~~end up~~ [4] wider breach width than one with scour hole.

- Extreme B: Use a STP postulated breach width of 4745 ft in the FSAR Revision 0, then use FLDWAV to estimate Qp and Tp. This may be the case where a bounding MLM breach width equation is used.

- High Qp: Use an average of 5 high Qp's estimated using empirical equations (Qp=336 kcfs).

As before, DBF levels were computed by Equation 6.6-1 with estimated Qp and Tp. The table below lists the breach parameters as well as estimated DBF levels for each scenario. The result of this estimation shows that the extremely conservative DBF level would be about 47 ft MSL, where as FSAR Revision 0 DBF level estimate based on the simulation of the Delft3D model was is 47.6 ft MSL.

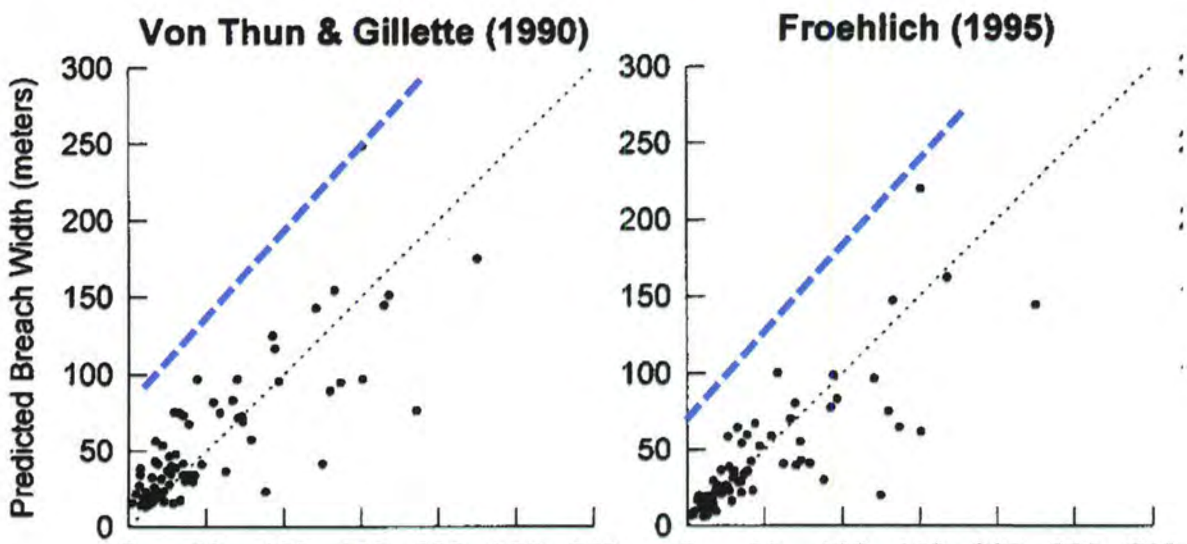

Figure 6.6-9. The best-fitted and envelop regression lines in predicting breach width (after Wahl, 1998).

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Table 6.6-4. DBF levels estimated by Equation 6.6-1 with extremely conservative MCR breach scenarios.

Run ID	B (ft)	Tf (hr)	Qp (kcfs)	Tp (hr)	DBF level H (ft MSL)	Remark
MLM-B-D0	1736	2.54	380	1.55	45.89	0 ft hole
Extreme B	4745	2.54	430	1.3	46.77	0 ft hole
High Qp	-	1.7	336	1.3	45.12	MLM envelop Tf

6.7 Frequency of Occurrence of MCR Breach

FSAR 2.4 dam breach analysis is based on a deterministic approach that does not consider the condition of the ~~levee~~ frequency of failure occurrence. However, this section includes an investigation of the probability of the MCR breach failure as referencing information. Generic dam failure frequency which is mainly based on small or medium size dams (length of less than 2000 ft) cannot be applicable to MRC breach. The above generic failure probability cannot be used because a total MCR ~~levee~~ length is 12.6 miles. The MCR breach failure frequency could be higher than the generic one. This report uses the following three sources of information:

- Foster et al. (2000: Table 5) said that an average annual frequency of embankment levee failure (earth-fill levee with filter) due to piping is about 1.89×10^{-4}/year for first 5 years of operation and 0.37×10^{-4} /year after 5 years of operation.
- The frequency of levee failures for the California Delta Levee System (a total levee length of 911 miles) is about 1.18×10^{-4}/year/levee mile.
- The frequency of the Suisun Marsh levee (a total of 75 miles) is about 4.76×10^{-4}/year/levee mile.

In summary, the estimated frequency of MCR ~~levee~~ failure is an order of **10^{-4}/year.** However, this estimate needs to be corrected considering the length and condition of the MCR ~~levee~~.

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For rare events, such as dam failure, frequency of failure is not meaningful. Probability of failure is more appropriate.

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embankment

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MCR

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embankment

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Is this estimate the author's opinion? The author does not provide any supporting references or additional analyses.

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embankment

Number: 8 Author: D3K890 Subject: Highlight Date: 7/15/2011 4:38:38 PM -04'00'

Unsure what the author means by "condition" of the MCR embankment.

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embankment

7 Conclusion

The author of this report performed a re-analysis of the main cooling reservoir (MCR) breach flood as part of reviewing the South Texas Project combined license application. The result of the re-analysis proves that both STP's and PNNL's MCR breach flood analyses are inaccurate and non-conservative~~.~~ [1]~~they~~ [2]nderestimated the MCR breach parameters (breach width and peak outflow rate) and the resulting design basis flood level. The author's re-analysis reveals that the reasonably conservative design basis flood level would be about 45 ft mean sea level, or it could exceed about 47 ft mean sea level if a very conservative MCR breach scenario is considered. Because the design basis flood level is critical in safety-related structural designs and flood protection, the author recommends revising the dam breach sections of FSAR and the current version of the SER using a correct and conservative MCR breach scenario.

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They

References

- Acrement G. J. and V. R. Schneider, 1984, Guide for Selecting Manning's Roughness Coefficients for Natural Channel and Flood Plains, USGS Water Supply Paper 2339.

- Baecher, G.B., M. Elisabeth, and R. D., Neufville, 1980, "Risk of Dam Failure in Benefit-Cost Analysis, Water Resources Research, Vol. 16, No. 3, Pages 449-456.

- Brunner, G. W., 2007, Using HEC-RAS to Perform a Dam Break Analysis, a paper distributed at the FY07 Training for Hydrology and Hydraulics for Dam Dafety Studies by USACE/ HEC, Davis, CA.

- Chauhan, S.S., D.S. Bowles, and L.R. Anderson, 2004, ASDSO Proceeding of Dam Safety, "Do Current Breach Parameter Estimation Techniques Provide Reasonable Estimates for Use in Breach Modeling?" 2004 ASDSO Proceeding of Dam Safety, Utah State University and RAC Engineers & Economics, Logan, UT.

- Chow, V.T. 1959. Open Channel Hydraulics. McGraw-Hill, New York.

- FLD-2D Software, Inc., 2006, FLO-2D User Manual, Version 2006.01.

- Foster, M., R. Fell, and M. Spannagle, 2000, "The Statistics of Embankment Dam Failures and Accidents", Can. Geotech. J. 37:1000-1024.

- Fread, D.L., 1988, The NWS-DMABRK Model, Quick User Guide, Revision 4, 1991, Hydrologic Research Laboratory, Office of Hydrology, National Weather Service, NOAA

- Fread, D.L. 1991. BREACH: An Erosion Model for Earthen Dam Failures. Hydrologic Research

- Fread, D.L., 1988 (revised in 1991), BREACH, An Erosion Model for Earthen Dam Failures, National Weather Service, National Oceanic and Atmospheric Administration, Silver Spring, MD.

- Fread, D.L., 1993, NWS FLDWAV Model: The Replacement of DAMBRK for Dam-Break Flood Prediction, Dam Safety '93, Proceeding of the 10th Annual ASDSO Conference, Kansas City, Missouri.

- Fread, D.L., 2000. NWS FLDWAV Model: Theoretical Description," Hydrologic Research Laboratory, Office of Hydrology, National Weather Service, NOAA (Revision 4 1991).

- Froehlich, D. L. 1995a. Embankment dam breach parameters revisited. Water Resources Engineering, Proceedings of the 1995 ASCE Conference on Water Resources Engineering, New York, 887–891.

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- Froehlich, D. C. 1995b. Peak outflow from breached embankment dam. Journal of Water Resources Planning and Management, American Society of Civil Engineers, 121(1), 90–97.

- Gee, D.M., 2008, Comparison of Dam Breach Parameter Estimators, HEC Report, U.S. Corps of Engineers Hydrologic Engineering Center, Davis, CA.

- Gee, D.M., 2010, Use of Breach Process Models to Estimate HEC-RAS Dam Break Parameters, 2nd Joint Federal Interagency Conference, Las Vegas, NV.

- Hydrologic Engineering Center (HEC), 2008, HEC-RASRiver Analysis Sytem, User's Manual, Version 4.0, USACE, Davis, CA.

- MacDonald, T. C., and Langridge-Monopolis, J. 1984. Breaching characteristics of dam failures. Journal of Hydraulic Engineering, 110(5), 567–586.

- Nagy, L., 2006, Estimating Dyke Breach Length from Historical Data, Periodics Polytechnica Ser. Civ. Eng. Vol 50, No. 2, pp125-139.

- Pacific Northwest National Laboratory, 2011, Calculation Worksheet: Confirm MCR embankment breach flood discharge and its sensitivity to breach parameter, submitted to NRC as a supplemental document for SER, Richmond, WA.

- SFWMD (South Florida Water Management District), 1980, Interim Final Draft Report on Embankment Failure Florida Power $ Light Company Martin Plant Cooling Reservoir, SFWMD, West Palm Beach, FL.

- STP (South Texas Nuclear Operating Company), 2007, South Texas Combined License Application, Revision 0, Part 2, Final Safety Analysis Report, Bay City, TX.

- STP (South Texas Nuclear Operating Company), 2009, South Texas Combined License Application, Revision 2, Part 2, Final Safety Analysis Report, Bay City, TX.

- STPEGS, 2006, Updated Final Safety Analysis Report (UFSAR for Units 1 & 2, Revision 13.

- U.S. Army Corps of Engineers, 2005, User's Guide to RMA2 WES, Version 4.5, Coastal and Hydraulics Laboratory, Waterway Experiment Station, Engineering Research and Development Center, Vicksburg, MS.

- U.S. Army Corps of Engineers, 2006, "Engineering and Design: Reliability Analysis and Risk Assessment for Seepage and Slope Stability Failure Modes for Embankment Dams", Department of Army, U.S. Army Corps of Engineers, ETL 1110-2-561, Washington, D.C.

- United States Geological Survey 2009. Water Resources of Illinois: n-Values Project. URL: http://il.water.usgs.gov/proj/nvalues/equations.shtml?equation=05-strickler.

This page contains no comments

- U.S. Bureau of Reclamation 1982. Guidelines for defining inundated areas downstream from Bureau of Reclamation dams, Reclamation Planning Instruction No. 82-11, U.S. Department of the Interior, Bureau of Reclamation, Denver, Colorado, 25p.

- U.S. Bureau of Reclamation 1988. Downstream hazard classification guidelines. ACER Technical Memorandum No. 11, U.S. Department of the Interior, Bureau of Reclamation, Denver, Colorado, 57p.

- U.S. Bureau of Reclamation, 1998, Prediction of Embankment Dam Breach Parameters: A Literature Review and Needs Assessment", DSO-98-004, Dam Safety Research Report, U.S. Department of the Interior, Bureau of Reclamation, Dam Safety Office.

- URS, 2007, "Technical Memorandum Delta Risk management Strategy (DRMS) Phase 1: Topical Area Levee Vulnerability Draft 2, URS Corporation/J.R. Benjamin & Associates, Inc., Oakland, CA.

- Von Thum, J.L. and D.R. Gillette, 1990, Guidance on Breach Parameters, unpublished internal document, U.S. Bureau of Reclamation, Denver, CO.

- Wahl, T.L. 2004. Uncertainty of Predictions of Embankment Dam Breach Parameters. Journal of Hydrology, 130(5), 389-397.

- Wahl, T.L., 1997, Predicting Embankment Dam Breach Parameters – A Needs Assessment, XXVIIth IAHR Congress, San Francisco, CA.

- Wahl, Tony L. , Gregory J. Hanson, Jean-Robert Courivaud, Mark W. Morris, René Kahawita, Jeffrey T. McClenathan, and D. Michael Gee, 2008. Development of Next-Generation Embankment Dam Breach Models. U.S. Society on Dams, 2008 Annual Meeting and Conference, April 28-May 2, Portland, Oregon, [online paper].

- Zhang X. and Wang, G, 2001, Flow Analysis and Scoring Hole Computation of Dyke-Breach," Proceeding of the International Association for hydraulic Researchers XXIX Congress, Theme E. Tsinghua University, Beijing, China.

Attchment 10c: STP 2.4.5 (summary of the history of the probable maximum storm surge review)

Probable Maximum Storm Surge Review and Analysis Performed By Staff

The staff reviewed probable maximum storm surge (PMSS) modeling analysis performed by the applicant. The applicant used three numerical models for their analysis: SURGE, SLOSH, and ADCIRC. Ultimately, the applicant based its PMSS flood determination on the results of ADCIRC. The staff's analysis involved confirmatory and independent analysis based on use of the SURGE and SLOSH models but ultimately the staff used SLOSH results only in its review.

Over the course of the staff's preparation of the SER Section 2.4.5 , the staff issued several RAIs. RAI 02.04.05-1 requested that the applicant provide the SURGE code. RAI 02.04.05-2 requested that the applicant provide HEC-RAS input and output files to estimate backwater effects corresponding to the PMSS estimates using the SURGE model. RAI 02.04.05-3 requested that the applicant provide an explanation of the selection of wind-stress correction factors using the surge analysis. RAI 02.04.05-4 requested that the applicant explain how the SLOSH results were extrapolated to probable maximum hurricane (PMH) conditions, describe whether the PMH conditions were consistently incorporated into the SURGE and SLOSH analysis, and explain how SLOSH was used in a conservative manner. RAIs 02.04.05-5 and 02.04.05-6 requested that the applicant explain why the PMH determined from NOAA NWS 23 was not used to run the SLOSH model to estimate the PMSS. RAI 02.04.05-6 requested that the applicant describe efforts to incorporate parameters of recent hurricanes into the selection of PMH parameters. RAI -02.04.05-7 requested that the applicant provide and assessment of seismically induced seiches in the main cooling reservoir. RAI 02.04.05-8 requested clarification on backwater calculations made by the applicant. RAI 02.04.05-9 requested that the applicant provide further explanation of the conservative of the SLOSH extrapolation methods that was used. RAI 02.04.05-10, RAI 02.04.05-11 and it's supplement requested that the applicant further clarify the conservatism in the applicant's use of SLOSH results, to provide further detail on the applicant's use of ADCIRC, and to conduct sensitivity analysis to support the applicant's conclusion.

The staff's activities other than the review of the FSAR and RAI responses included independent determination of the PMH parameters, confirmatory simulations using SURGE, confirmatory calculations of the applicant's SLOSH extrapolation, and independent application of SLOSH to estimate the PMSS using the PMH. These activities are briefly described below.

The staff used NWS 23 (NOAA 1979) to independently estimate the PMH for the STP site. The staff estimated that the maximum wind speed for a moving and a stationary hurricane at the STP site would be approximately 70.5 and 66.9 m/s (157.6 and 149.7 mph), respectively. The staff-estimated stationary hurricane wind speed of 149.7 mph is consistent but slightly lower than the applicant's estimated range of 68.0 to 71.5 m/s (152 to 160 mph) reported in FSAR Table 2.4S.5-3 (STPNOC 2007).

The applicant provided the SURGE code and input files to the staff. The staff performed an independent analysis using the applicant's implementation of the SURGE model and confirmed the applicant's analysis. Because the results from SURGE were not the applicant's most conservative assessment, the staff did not undertake a further analysis using the SURGE model.

The applicant developed a regression equation using the NOAA pre-computed Categories 1 through 5 SLOSH maximum of maximum (MOM) values using the central pressure differential as the predictor of surge elevations. The applicant provided the regression procedure and described its use. The staff verified the applicant's results using the PMH pressure differential. The staff determined that the applicant's extrapolation based on the NOAA pre-computed the SLOSH Categories 1 through 5 MOM may not have yielded conservative estimates of peak water levels at the site, because there is no physical basis for choosing the extrapolation equation that the applicant used. Therefore, the staff independently estimated the PMH water surface elevations at the STP site using the SLOSH model input with PMH parameters and identified that the surge level simulated by the SLOSH model is higher than the applicant's initial estimate by the SURGE model.

The staff used SLOSH to independently estimate surge levels. The staff used PMH parameters as recommended by NOAA NWS 23. The staff set the radius to maximum winds, the approach angle, and the forward speed to vary within the range recommended by NWS 23. Three landfall points were selected such that that the first landfall point was located at a distance equal to the radius of the maximum winds west of the mouth of the Colorado River Navigation Channel at the barrier islands; the second point was centered on the mouth of the Colorado River Navigation Channel at the barrier islands; and the third was located a distance equal to the radius of the maximum winds east of the mouth of the Colorado River Navigation Channel at the barrier islands. All PMH storm tracks used by the staff were straight. There were 81 combinations of these parameters. The staff also added a vertical upward shift to the initial water surface in SLOSH to account for a 10-percent exceedance tide, initial rise, and 100-year sea-level rise.

The staff's SLOSH simulations indicated that the maximum storm surge water surface elevation near the STP Units 3 and 4 site would be produced by a large (in terms of radius to maximum winds), fast-moving (in terms of forward speed) storm that would produce prevailing winds blowing from the east toward the STP Units 3 and 4 site. The staff estimated that the storm surge maximum water surface elevation at the STP Units 3 and 4 site is located would probably be between 11.3 and 11.6 m (37 and 38 ft) NGVD29. To account for wind-wave action, the staff estimated the wind-wave amplitude to be 0.27 to 0.36 m (0.9 to 1.2 ft) following the methods in the Coastal Engineering Manual (USACE 2008). The staff determined that the corresponding conservative wave runup to be approximately 0.20 m (0.65 ft). Therefore, an evaluation of wave action shows that it adds 0.47 to 0.56 m (1.55 to 1.85 ft) to the peak level of inundation estimated by the staff's SLOSH simulations. The staff estimated the maximum PMSS water surface elevation to be between approximately 11.8 to 12.2 m (38.6 to 39.6) NGVD29, including the effects of wind waves at the STP Units 3 and 4 site.

The staff determined that ADCIRC was an appropriate model to simulate storm surges from hurricane events based on the response received . The staff concluded that bathymetry features and near-shore topography data incorporated into the ADCIRC model application by the applicant provided more detailed site-specific information for storm surge simulation at the STP site. In the response to RAI 02.04.05-11, the applicant stated that extensive validation of ADCIRC predictions on the Texas coastline for historical hurricanes has been carried out. The applicant notes that these validation studies included Hurricanes Rita and Ike that produced large storm surges, and accurate measurements of hurricane properties and surge levels for these storms are available. The applicant also stated that along the coastal areas of the United States, the FEMA has certified ADCIRC for use in development of Flood Insurance Rate Maps that need to account for flooding from storm surges. The applicant also states that ADCIRC is the standard coastal model used by the USACE. The staff's independent review found that the USACE ADCIRC model has a long history of development, verification, and validation (Luettich and Westerink 1992; Luettich et al. 1998; Luettich et al. 1992; Westerink et al. 1992; Blain et al. 1994; Grenier et al. 1994; Westerink et al. 1994; Gica et al. 2001; Dietsche et al. 2007; Demirbilek et al. 2008; Westerink et al. 2008; Funakoshi et al. 2009; Bunya et al. 2010). The staff has concluded that the applicant has appropriately selected a conservative PMH scenario to simulate using ADCIRC. The staff concluded that the applicant's ADCIRC simulations for determination of PMSS at the STP site were adequate.

Attchment 11a: Form 757 Attachment 1 – final2 (reasons for non-concurrence)

Attachment 1: Reasons for Non-Concurrence

Title: South Texas Project Combined License Application Review: SER with no Open Item Chapter 2.4
Name: Dr. Hosung Ahn
Date: June 8, 2011

<u>Summary</u>

This document addresses three non-concurring issues on the Safety Evaluation Report (SER) for the South Texas Project (STP) Combined License Application (COLA). The non-concurring individual, Dr. Hosung Ahn, is hydrology lead for reviewing the STP COLA FSAR Chapter 2.4 Hydrology. I am filing this non-concurrence package as a SER document contributor who is not on the list of document concurrences (ML1114507490 for concurrence sheet and ML1114500734 for SER).

My concerns are related to determining two site parameters - design basis flood level and maximum groundwater level - which are important site parameters for structural design and flood protection of safety-related facilities. It is my opinion that the staff's review of these two parameters was not properly done. Specifically, the postulated main cooling reservoir breach scenario and design basis flood level estimation in the SER are neither accurate nor conservative. To justify my technical position and to provide a potential solution, I performed a re-analysis of design basis flood. The full re-analysis report will be provided as Enclosure 1 to this package.

The applicable regulations in estimating these two site parameters are: (a) 10 CFR 52.79(a)(1)(iii) and GDC 2 of Part 50, as they relate to identifying hydrologic site characteristics with appropriate consideration of **the most severe** of the natural phenomena; and (b) 10 CFR 100.20(c)(3), as it requires to identifying and characterizing the **maximum probable** hydrologic events such as wind speed and precipitation for site safety analysis. The following statements describe detailed concerns on each issue, followed by a proposed solution.

<u>Issue #1: Main Cooling Reservoir (MCR) Breach Flood Analysis in SER Section 2.4.4</u>

1) ABWR DCD Tier 1 requires that the design basis flood level be less than 1 ft below the plant grade. STP identified that a postulated MCR levee breach scenario will create design basis flood that exceeds the DCD limit – It is handled as a departure in FSAR. To formulate a breach scenario, STP estimated width and time of levee breach based using two empirical regression equations: Froelich equation for breach width and MacDonald and Langridge-Monopolis (MLM) equation for breach time. In my opinion, STP's approach is incorrect because (a) it is a mix-and-match approach (use two different equations for one breach scenario) which is not acceptable technically; (b) The Froehlich breach width equation,

which is good for the breach widths less than 164 ft, is not applicable to the MCR breach which is much longer than 164 ft in length; and (c) STP excluded the MLM breach erosion volume equation that may result in the most conservative breach width. Especially, STP justified in a RAI response that the MLM equation is not for breach width. It is my opinion that the STP's justification is incorrect, because the MLM breach erosion volume equation that can be used to estimate breach width (e.g., dividing the erosion volume by levee cross section area) has been classified and used widely for estimating breach width. My re-analysis report addresses this issue.

2) The Pacific Northwest National Laboratory (PNNL) wrote a draft SER. To postulate MCR breach, PNNL simulated breach outflow hydrographs using the BREACH model. However, PNNL set up the BREACH model incorrectly: They set up the BREACH model to limit levee width of 1000 ft and tailwater bottom width of 600 ft so that the simulated breach width and breach outflows are constricted by unrealistically small levee and channel cross sections. This constriction results in a less conservative design basis flood level. Enclosure 2 shows the corrected result of from my analysis. My re-analysis report presents a comprehensive MCR breach analysis considering the above correction as well as the effects of scouring hole, cement blocks, and levee's sand core on the breach process (Enclosure 1). The PNNL modeling has serious deficiencies, so it cannot be accepted as a credible safety analysis.

3) PNNL simulated the BREACH model with three surface roughness coefficient values (n-value of 0.025, 0.05, 0.075), then picked the case of 0.075 as a postulated breach scenario. However, PNNL never justify the selection of the n-value of 0.075. My re-analysis shows that n-value could be higher than 0.075.

4) PNNL confirmed the postulated breach scenario using an actual breach case from the Martin Cooling Pond in Florida, but this process is highly subjective compared to the statistical regression methods that used multiple historical data points to come up the best-fitting (or bounding) regression line of breach parameters. They also estimated breach parameters using selected empirical equations; however they do not provide the reasons why the conservative breach parameter estimates using empirical equations were ignored in SER.

5) The BREACH model is based on the Meyer-Peter & Muller (MPM) sedimentation equation. However, many hydrologists have noted that the MPM method is not adequate for dam breach conditions as it is developed for estimating sedimentation in small mountain areas. The MCR breach condition is quite different from the mountain condition in terms of slope and bottom condition. Because the result of BREACH model is highly depending on input parameters, many researchers discredit the BREACH model. Even the author of the BREACH model recommended using the BREACH model as a supplemental purpose only. PNNL's approach to estimate breach parameters using the BREACH model results in a biased and less conservative breach scenario, as my analysis demonstrates.

6) PNNL's BREACH simulations did not consider the effects of the scouring of the foundation during the MCR breach process. Proper consideration of the scouring effects will substantially increase the breach outflows and the resulting flood level at the site as the attached re-analysis report shown. In summary, the postulated MCR breach scenario and the resulting DBF flood level estimates in SER are inaccurate and non-conservative.

Issue #2: Flood Analysis of Hurricane and MCR Breach Combination in SER Section 2.4.5

1) At the beginning of the FSAR review, the staff noted that combined events such as a combination of storm surge and MCR breach, could create a more severe flood than that of a MCR breach only. In this regard, PNNL reviewed the applicant's storm surge modeling and concluded that the applicant's probable maximum hurricane (PMH) used in the surge model is conservative. It is my opinion that the STP's PMH parameters are not conservative because the applicant's PMH is based on the NOAA's manual (NWS-23) which was developed in 1979 but has not been updated for the storms observed during the past 30 years. It is generally known that the intensity and frequency of hurricanes have been increased during the past decades. SER does not address this change adequately.

2) PNNL concluded in SER Section 2.4.5 (p. 81) that the STP's ADCIRC simulations for determining the storm surge level at the site is adequate. However, they provide no background information or any justifications on that conclusion. PNNL never review the input and output of the STP's ADCIRC model nor conduct any independent confirmatory modeling other than SLOSH simulations which were discarded by the applicant. The main reason for discarding the SLOSH model is that the STP's SLOSH model is less accurate than STP's ADCIRC model that uses finer grids with detailed topography data. Since STP adopted to use the ADCIRC model, NRC should review the STP's ADCIRC model to support the conclusion of adequacy of surge modeling and to justify the reasons why the combined event of hurricane and MCR breach is discarded for consideration.

Issue #3: Maximum Groundwater Level in SER Section 2.4.12

1) ABWR DCD Tier 1 specifies the requirement of the maximum groundwater level below two feet plant grade. This DCD parameter is used in structural design and the NRC reviewer must check whether the site groundwater levels exceed the DCD limit or not. The STP FSAR adopts a site characteristic for a maximum groundwater level of 28 ft MSL under the proposed plant grade of 34 ft MSL. STP proposed to install a 2-ft clay layer or cap immediately below grade level to interrupt infiltration from surface ponding to groundwater body through backfill materials and to maintain the maximum groundwater level. STP also proposed a stone layer to prevent erosion of the clay layer from MCR breach flood, so that there is no interaction between surface and ground waters to meet the DCD condition.

2) However, the NRC staff assumed conservatively that the clay cap could be eroded away during the MCR breach flood, resulting in a saturation condition of the entire vertical profile from the plant grade to the level of 28 ft MSL. This condition creates a departure of the DCD condition, which is different from what STP assumed in the FSAR. The NRC SER states that this groundwater condition is included in the engineering evaluation as discussed in SER Section 3.8, but it is not confirmed nor committed yet. It is author's opinion that the

SER should describe why the stone layer is breakable and that FSAR should state this departure condition, including properly reflecting the departure condition in structural designs and flood protection measures. So far, staff has not informed this departure condition to STP. It is my opinion that the process of reviewing and handling the maximum groundwater level in SER is inadequate for the above reasons. The saturation condition should be handled as a DCD departure.

<u>Proposed Solution</u>

It is my opinion that both FSAR and SER contain serious errors in the MCR breach flood analysis. The analysis is also not conservative to meet the requirements of NRC regulation as my attached re-analysis report demonstrated. The result of my re-analysis shows that the design basis flood level could be much higher than the STP's estimate. Therefore, I am recommending with this non-concurrence process that SER should be revised before presenting it to the ACRS and ASLB.

Enclosure 1: Re-analysis of MCR Breach Flood Analysis
Enclosure 2: PNNL's Calpack – commented and corrected version by Hosung Ahn

Attachment 11b: Enclosure 1 – (Re-analysis of MCR Breach Flood)

Enclosure 1: Re-Analysis of MCR Breach Flood

(Attachment to the Non-concurrence on the Safety Evaluation Report Section 2.4.4 for the South Texas Project Units 3&4 Combined License Application)

June 20, 2011

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U.S. Nuclear Regulatory Commission

NRO/DSER/RHEB

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1 Executive Summary

This document addresses one of the three non-concurring issues raised by Dr. Hosung Ahn on the Safety Evaluation Report (SER) Chapter 2.4 for the South Texas Project (STP) Combined License Application (COLA). This report address the issue related to determining a site parameter - design basis flood level - which is critical for structural design and flood protection of safety-related facilities.

STP provided in their Final Safety Analysis Report (FSAR) the estimation of the design basis flood (DBF) level caused by a postulated main cooling reservoir (MCR) breach. NRC staff reviewed the estimation of the DBF elevation and concluded in the SER that the applicant's DBF estimation is acceptable. PNNL help the NRC staff in reviewing the STP's MCR breach analysis. PNNL performed a breach modeling as a confirmatory analysis, however their breach modeling includes a serious error. It is my opinion that the postulated main cooling reservoir breach scenario and design basis flood level estimation in both FSAR and SER are neither accurate nor conservative. To justify my technical position and to provide a potential solution, I performed a re-analysis of the MCR breach flood.

The applicable regulations in estimating the design basis flood level are: (a) 10 CFR 52.79(a)(1)(iii) and GDC 2 of Part 50, as they relate to identifying hydrologic site characteristics with appropriate consideration of **the most severe** of the natural phenomena; and (b) 10 CFR 100.20(c)(3), as it requires to identifying and characterizing the **maximum probable** hydrologic events such as wind speed and precipitation for site safety analysis.

This report includes all available relevant information on the MCR breach analysis and modeling. The analysis done in this report is very comprehensive – It is a multiple-step process including the following modeling:

- Levee breach parameter estimation using all available empirical equations
- Validation of breach parameters and outflows using the BREACH model
- Simulating breach outflow using the FLDWAV model, and
- Routing of breach flood at the site using the FLO-2D model.

The result of the breach and flood modeling indicates that the reasonably conservative DBF levels at the proposed Units 3 & 4 would be about 45 ft MSL considering a enough margin for the wind wave setup and modeling uncertainty. This new estimate is much higher than the STP's DBF estimation of 40 ft MSL. As this increase in flood level is significant in structural design and flood protection measures, the author of this report strongly recommend revising both the FSAR and the current version of the SER.

In addition to site specific data, this report includes a lot of useful information in relation to the levee breach flood analysis in general. However, the readerships of this report should note that the content of this report could be applied to other levee breach flood analyses but not directly used because the levee breach flooding is governed by many site-specific parameters and conditions.

2 Introduction

Final Safety Analysis Report (FSAR) Section 2.4.4 of the South Texas Project (STP) combined licensing application (COLA) addresses potential dam failures. The purpose of the dam failure analysis in FSAR is to ensure that potential external flood hazard to safety-related structures due to failure of dam is considered in the plant design and flood protection measures. This FSAR section covers postulation of dam failure scenarios, hydrodynamic analysis of dam failures, and determination of flood characteristics (e.g., flood level, duration, velocity, etc.) at the site. The dam failure flood analysis used in FSAR 2.4.4 is a deterministic approach – We assume a dam failure without considering the condition of the dam or probabilities of failure occurrence and consequence.

The dam failure analysis in the STP's FSAR includes the potential main cooling reservoir (MCR) embankment breach and the effects of the breach on the site flooding. STP identified that the MCR breach is the controlling design basis flood (DBF) scenario among all potential flood events, including rainfall, hurricane, tsunami, and dam breaches. They also identified that the controlling flood level is higher than the bounding design flood level of one feet below the plant grade specified in the ABWR Design Certificate Document (DCD), thus handled the DBF level as a DCD departure. The ABWR DCD requires that the maximum flood level should be 1 ft below the plant grade.

The MCR covers an area of approximately 3 miles by 4 square miles and is formed by an embankment levee about 12.4 miles long. The area of the MCR is about 7,000 acres. For the Units 3&4, STP proposed to use the MCR which was originally built for four reactor units. The MCR was designed for the normal maximum operation level of 49 ft mean sea level (MSL). However, STP has been operating the reservoir at a pool level of 45 ft MSL after a field seepage test in early 1990s. With the proposed two units, STP proposed to raise the operating reservoir level from 45 ft MSL to 49 ft MSL. The raise of the operating level could increase the potential of MCR levee breach which is a significant safety issue.

The STP's estimation of the DBF elevation caused by a postulated MCR breach scenario is 40 ft above MSL. The breach scenario is based on the estimated breach width of 417 ft and breach time of 1.7 hours.

The NRC staff assisted by the PNNL completed the review of the STP's MCR breach analysis in May 2011, and concluded that the applicant's analysis of the MCR breach flood is acceptable (refer to the SER Section 2.4.4). However, the author of this report (Dr. Hosung Ahn) found that the analyses of MCR breach flood by both STP and PNNL are inaccurate and non-conservative. Therefore, the author made a re-analysis of the MCR breach flood in a conservative manner. This report summarizes the result of the re-analysis and findings. Section 3 of this report summarizes the regulatory requirement of dam failure analysis for reactor licensing. Sections 4 and 5 discuss the problems in STP's and PNNL's dam breach analyses, followed by the re-analysis of MCR breach flood in Section 6.

3 Regulatory Basis

The applicable regulatory requirements for analyzing the effects of dam failures on the proposed new reactors are as follows:

- 10 CFR Part 100, as it relates to identifying and evaluating hydrological features of the site. In site evaluation, 10 CFR 100.20(c) specifies the requirement to consider physical site characteristics, including seismology, meteorology, geology, and hydrology.

- 10 CFR 100.23(d), as it sets forth the criteria to determine the siting factors for plant design bases with respect to seismically induced floods and water waves at the site.

- 10 CFR 52.79(a)(1)(iii) and GDC 2 of Part 50, as it relates to identifying hydrologic site characteristics with appropriate consideration of **the most severe** of the natural phenomena that have been historically reported for the site and surrounding area and with **sufficient margin** for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

In addition, the NRC staff has been used appropriate regulatory positions of the following regulatory guides for the acceptance criteria of dam failure analyses:

- RG 1.27, "Ultimate Heat Sink for Nuclear Power Plants"

- RG 1.59, "Design Basis Floods for Nuclear Power Plants," as supplemented by best current practices

- RG 1.102, "Flood Protection for Nuclear Power Plants."

Specifically, Part 100.20(c)(3) states that the **maximum probable** wind speed and precipitation must be identified and characterized in the analysis of reactor site safety. Correspondingly, the Regulatory Position 1.c in RG 1.59 specifies the requirement of analyzing dam failures to identify **the worst** site-related flood at a nuclear power plant. Section 2.4.4 of NUREG–0800 (SRP) provides technical rationale and acceptance criteria for the analysis of dam failures. In this SRP section, the terms 'conservative' is used 8 times and the term 'the most severe' is appeared 7 times in order to emphasis the conservatism that must be applied to the analysis of dam failures. The author of this report claims that the dam failure analyses by both STP and PNNL do not meet these regulatory requirements.

4 MCR Breach Analysis Submitted by STP

STP presented the main cooling reservoir (MCR) breach flood analysis in the FSAR Section 2.4.4 and the responses of relevant RAIs. The MCR is a manmade, in-ground reservoir enclosed by about 12.4 miles embankment levee. The function of the MCR is to supply water to dissipate the heat generated from the nuclear power plants. STP analyzed the onsite flood hazard resulting from a postulated breach of the north MCR embankment levee located about 2,300 ft south from the proposed Units 3 and 4.

The MCR embankment levee was built with 40-feet-high rolled soil materials above the clay-sand-silt top surface. The interior face (reservoir side) of the dyke is lined with 2 ft thick soil cement blocks to absorb wave actions. The outside face of the levee is covered by grass on bare soils (refer to Figure 4-1). The design normal maximum operating water level in the MCR is 49 ft MSL. STP postulated the MCR dyke failures caused by the excessive seepage from: (1) piping through the foundations of embankment, (2) seismic activity leading to liquefaction of the foundation soils, and (3) erosion of the embankment due to overtopping or wind-wave events.

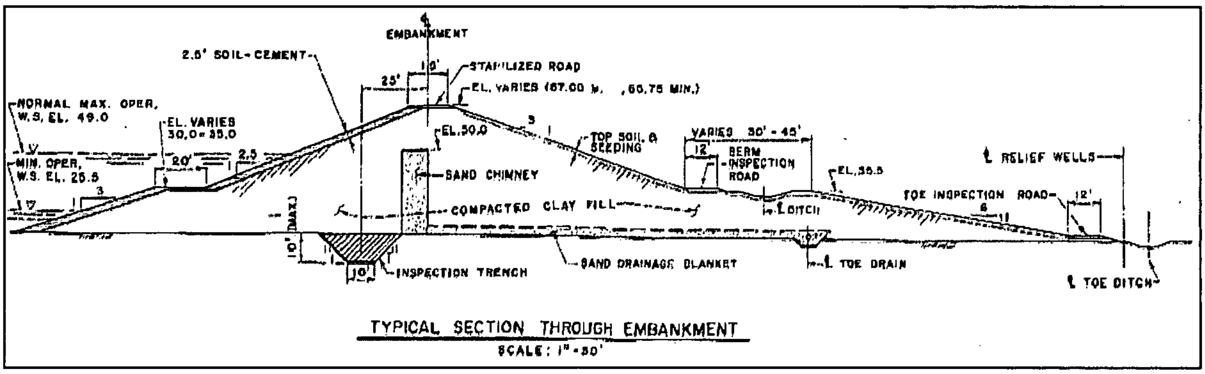


Figure 4-1. Cross section of the MCR embankment levee (provided by STP)

STP postulated and analyzed a northern MCR levee breach that results in the most severe flood (in terms of the flood level) at the proposed site. The following summarizes the STP's MCR breach flood analysis:

(1) STP estimated levee breach parameters (e.g., width and time of breach) based on the two empirical equations: Froelich equation for breach width and MacDonald and Langridge-Monopolis (MLM) equation for breach time. STP validated these parameter estimates using the NWS BREACH model (Fread, 1991; or just BREACH hereafter).

(2) STP simulated the MCR breach outflow hydrograph using the FLDWAV model (Fread and Lewis, 1998) with the above estimated breach parameters.

(3) STP estimates the flood characteristics (e.g., flood level, time, velocity, etc) at the plant sites using the RMA2 model (Donnell et al., 2008) with the FLDWAV breach outflow hydrograph.

Specifically, STP used empirical equations to estimate dam breach parameters for earth-filled structures as recommended by the U.S. Bureau of Reclamation (USBR) (Wahl 1998). STP assumed that a service road immediately downstream of the toe of the MCR dyke will be the control bottom (29 ft MSL) for the postulated breach.

STP used the Froehlich equation to estimate the breach width. The estimated average breach width (e.g., top and bottom of the breach) is 417 ft. The STP estimated the time to failure of 1.7 hours using the MLM bounding equation. STP obtained the peak breach outflow of 62.600 cfs using the NWS BREACH model. They also estimated the peak breach outflow rate of 130,000 cfs using the FLDWAV model, which was used in the RMA2 model.

Evaluation of STP's Breach Analysis

It is the author's opinion that STP's breach analysis is not conservative in terms of its approach and model parameters. To the author, breach parameter estimates are of concerns as they are very sensitive to the resulting DBF level. The following bullets summarize this issue:

- STP used **a mix-and-match approach** in estimating MCR breach parameters – The MLM equation for time to peak and the Froehlich equation for breach width. This results in less conservative parameters.

- In estimating the MCR breach width, STP ignored the MacDonald and Langridge-Monopolis (MLM) equation (USBR, 1998) which results in the most wide breach width. In a RAI response, STP stated that the MLM equation is not for breach width. This statement is incorrect. The MLM equation which was originally developed for estimating breach eroding volume has been classified and used widely in estimating breach widths (for examples, Table 1 in Wahl (1997); Wahl (2003); Gee (2010); Brunner (2007); and many others).

- STP used the Froehlich's breach width equation (1995), however USBR (1998) states that the Froehlich's equation appears to be the best predictor for the breach widths less than 164 ft because the breach width records used to develop the Froehlich equation are small. In other words, Froehlich's breach width equation is not applicable to the MCR breach case.

- The breach parameters estimated by the BREACH model is very sensitive to the roughness condition (so called the Manning's n-value) of the breach bottom. The Manning's n-value of 0.045 used in STP's BREACH modeling is not conservative. Also, STP does not consider the erosion of the levee below the 29 ft MSL in their breach analysis. Considering these two factors will substantially increase breach width and outflows as demonstrated in Section 6 of this report.

- The set-up of both STP's and PNNL's BREACH model is incorrect. In reality, the downstream tailwater condition of the MCR breach is wide 2-dimentional overland flow. However, both STP and PNNL set a limited tailwater cross-section width of 600 ft and levee crest width of 1000 ft, which restricts the simulation of breach width and outflows. This constriction results in an underestimation of the DBF flood level – It will be discuss further in Section 6 of this report.

5 MCR Breach Analysis by PNNL

PNNL performed an independent confirmatory analysis of the MCR breach flood using the breach models developed by the applicant. The following summarizes the procedure used by PNNL:

(1) PNNL simulated breach widths and outflow hydrographs using the BREACH model with three Manning's n-values, then picked the most conservative one as a postulated breach scenario.

(2) They confirmed the breach scenario using empirical breach parameter equations as well as an actual breach case from the Martin Cooling Pond in Florida.

(3) PNNL used the applicant-provided RMA2 model to simulate flood characters at the site.

PNNL carried out a sensitivity analysis by varying some of the BREACH model parameters to determine the sensitivity of model input parameters (e.g., levee geometry, embankment material properties, headwater/tailwater conditions, etc.) on the breach parameters (e.g., width, time, peak flow) (PNNL, 2011). As a result of the BREACH sensitivity analysis, PNNL identified that the Manning's n-value that represents the bottom roughness condition is very sensitive to the breach parameters and outflows. They used the following three different n-values in the BREACH simulation:

- Simulation 1: n = 0.025
- Simulation 2: n = 0.050
- Simulation 3: n = 0.075.

Then, PNNL selected the Simulation 3 as the postulated breach scenario as it produces the largest breach peak outflow (130,000 cfs). The corresponding peak breach width and time are 515 ft and 1.99 hours.

PNNL also compared the BREACH-simulated outflow characteristics (peak flow and time, breach width) with the selected historical data from the database prepared by USBR (1998). From the review of the recorded breach events, PNNL selected a recorded breach event occurred in 1979 at the Martin Cooling Pond in Florida. The reservoir condition is similar to that of the MCR in terms of storage volume and water depth. The final average breach widths for the Martin Cooling Pond breach and the estimated MCR breach are 607 ft and 688 ft, respectively. Based on this comparison, PNNL concluded that their MCR breach parameters and flood estimates are reasonable.

<u>*Evaluation of PNNL's Breach Analysis*</u>

However, PNNL does not justify the selection of the n-value of 0.075 or why this value is conservative. Further, the author of this study found that PNNL's BREACH model to estimate the breach outflows and the sensitivity analysis is flawed by an incorrect model set-up. In

addition, validating the BREACH simulating result by a single arbitrarily selected breach record is not objective. The following summarizes the author's main concerns on the PNNL's analysis:

- PNNL's calculation package (2011: calpack) provides breach parameter estimates using three empirical equations (namely USBR, MLM, Froehlich) and the BREACH model. However, PNNL does not explain why MLM estimates that are the most conservative are excluded in further breach analysis. This is against general engineering practice or the guidance by federal agencies. Both USBR (1998) and USACE (2007) recommend using all available methods to determine breach parameters as the process involves a lot of uncertainty.

- PNNL used a faulty BREACH model setup by using a limited levee crest width of 1000 ft and tailwater bottom width of 600 ft (refer to Figure 5-1), which constrict the breach process and the resulting breach outflows. Finally, this modeling error results in an underestimation of breach width, outflows, and DBF flood level. This is an unacceptable modeling error. *(Note: The author notified this error to PNNL and NRC staffs in advance (February, 2011) of completing the SER, but my input has been ignored)*.

- The NWS BREACH model is based on the Meyer-Peter & Muller (MPM) sedimentation equation which results in a lowest sedimentation rate compared to other methods (refer to "Erosion and Sedimentation", by P. Julien, 2010). Many hydrologists pointed out that the MPM method is not adequate for dam breach conditions as it is developed for small mountain creeks.

- Because the result of BREACH model is highly depending on the model inputs, many hydrologists discredited the use of the BREACH model (e.g., Wahl, 1999). Also, the author (Dr. Fread) of the BREACH model recommends that BREACH model be used supplemental purpose only. PNNL's approach that relies only on the BREACH could be biased as this report demonstrated.

- Both STP and PNNL ignored the effects of sand core and clay cement blocks on the BREACH mode. Also they ignored the effects of scour hole on the breach process. Proper consideration of these factors will increase breach outflow and DBF level estimates substantially this report will be demonstrated.

The next section presents a re-analysis of MCR breach flood by considering the above concerns and by adding a reasonable conservatism on the estimation.

(a) PNNL's BREACH Input File for Simulation 3

MCR							
50.9	66.	29.	32.	0.	0.	.02	.8
30.	30.	30.	0.	0.	0.	0.	0.
0.	1.	100.	0.	0.	0.	0.	0.
7050.	7000.	6850.	5690.	2050.	0.	0.	0.
52.	49.	29.	25.5	20.	0.	0.	0.
28.	29.	32.	34.	40.	45.	50.	0.
0.	600.	1200.	1600.	2000.	2400.	2800.	0.
.06	.06	.06	.06	.06	.06	.06	0.
2.5	3.	0.	0.	0.	0.	0.	
0.	0.	0.	0.	0.	0.	0.	
0.001	.35	105.	.075	15.	200.	8.	
1.	16.	1000.	8.	0.	0.	0.	0.
.001	0.001	.1	120.	.01	1.	0.	

This small bottom tailwater section (600 ft) constricts the simulated breach width and outflows.

Again, this small levee crest length constricts the breach width and outflows.

Manning's n-value

(b) Hosung Ahn's BREACH Input File for Simulation 3

MCR Ahn's Run							
50.9	66.	29.	32.	0.	0.	.02	.8
30.	30.	30.	0.	0.	0.	0.	0.
0.	1.	100.	0.	0.	0.	0.	0.
7050.	7000.	6850.	5690.	2050.	0.	0.	0.
52.	49.	29.	25.5	20.	0.	0.	0.
28.	29.	32.	34.	40.	45.	50.	0.
0.	3000.	3000.	3000.	3000.	3000.	3000.	0.
.06	.06	.06	.06	.06	.06	.06	0.
2.5	3.	0.	0.	0.	0.	0.	
0.	0.	0.	0.	0.	0.	0.	
0.001	.35	105.	.075	15.	200.	8.	
1.	16.	3000.	8.	0.	0.	0.	0.
.001	0.001	.1	120.	.01	1.	0.	

Figure 5-1. Comparison of PNNL's BREACH input file and a corrected one by the author.

6 Re-analysis of MCR Breach Flood

This section presents the procedure, result, and findings of the MCR breach flood re-analysis which was performed to support the concerns raised by the author. The re-analysis is based on a comprehensive modeling approach. The author considered all applicable empirical breach equations and multiple breach models. As was done by the STP, the re-analysis used a deterministic dam breach analysis that does not consider the condition of the MCR levee or the frequency of the potential MCR levee failures. The author postulated multiple breach scenarios in a conservative manner. The errors found in STP's and PNNL's BREACH modeling were fixed here. This study also considers the effects of sand core, cement blocks, and scour hole on the breach progression and the resulting flood.

Because there is no model that can handle both breach and flood simulation simultaneously, the author adopted a multi-step approach in series. The procedure used in the re-analysis is summarized as following:

(1) This study includes an estimation of breach parameters using all applicable empirical equations in a comprehensive manner. The breach parameters include width and time of breach as well as peak breach outflows.

(2) The author used two physically-based breach models, namely NWS BREACH (or just BREACH hereafter) and FLDWAV, to obtain and validate the breach parameters estimated by empirical equations and to generate breach outflow hydrographs.

(3) The author used the FLO-2D model to simulate the overland flow at the proposed plant site using the FLO-2D hydrodynamic model.

Because the RMA2 model which was used by STP that is not available to the NRC staff, the author used the FLO-2D model which is another numerical 2-dimentional hydrodynamic flow routing software. The following figure compares the different modeling approaches used for the MCR breach flood analysis. The following subsections describe details of each step in the re-analysis.

<u>STP's Approach</u>
Empirical Equ's → Breach Width and Time → FLDWAV → Hydrograph → RMA2
Use BREACH to validate width and time

<u>PNNL's Approach</u>
BREACH --→ Hydrograph → RMA2

<u>Re-analysis Approach</u>
Empirical Equ's → Breach Width and Time → FLDWAV → Hydrograph → FLO-2D
BREACH-- → Hydrograph → FLO-2D

Figure 6-1. Comparison of MCR breach analyses by STP, PNNL, and the author.

6.1 Concerns on the MCR Breach Analysis

In general, dam breach parameters are very sensitive on the resulting flood levels. This is true for the MCR breach case as shown by both STP's and PNNL's studies. At the beginning of the STP COL license application, STP postulated a very conservative breach scenario that results in a conservative design basis flood (DBF) level. This initial breach scenario is even more conservative than that for Units 1&2. However, STP changed the original scenario by adopting a narrow breach width through the FSAR Revision 2. The table below summarizes the different breach scenarios.

Table 6.1-1. Comparison of different breach scenarios and resulting design basis flood (DBF) levels.

Source	Distance from levee to the site (ft)	Initial Reservoir Water Level (ft MSL)	Breach Width (ft)	DBF Level at the site (ft MSL)
UFSAR for Units 1 & 2	600	50.9	2000	50.8
FSAR Rev. 0 for Units 3 & 4	2300	50.9	4757	47.6
FSAR Rev. 2 for Units 3 & 4	2300	50.9	417	40
Victoria ESP	1000	93.9 ft NAVD88	2034	90.6 ft NAVD88 (~10 ft in depth)

Note: The estimated DBF levels for the STP FSAR Rev. 0 andVictoria ESP is based on the simulation of the Delft3D model, while that of STP Rev. 2 is by the RMA2 model.

The change of the STP's breach scenario triggered an attention to the NRC reviewers, resulting in numerous RAIs, site audits, and independent confirmatory analyses. However, the author believes that this issue has not been resolved in the SER in a satisfactory manner. It is the author's opinion that both STP's and PNNL's breach flood estimates are neither correct nor conservative. To help resolving this issue, the author performed a comprehensive re-analysis.

Specifically, it is my opinion that both STP's and PNNL's approaches are biased by using selective methods and by ignoring conservative equations or methods without proper justification. In general, it is recommended by many papers and federal guides that the engineers who practice dam breach analysis should consider a variety of methods as dam breach estimations contain a considerable uncertainty. For instance, Brunner (2007) in USACE provides the following recommendation:

> "In general, several methods should be used to predict a range of breach sizes and failure times. It is recommended that the user uses all empirical equations, such as the three recommended above: Froehlich (1995b), MacDonald and Langridge- Monopolis (1984), and Von Thun and Gillette (1990). Additional regression equations can also be used, as well as any of the physically based computer models (the BREACH model is currently recommended). This will lead to a range of values for the breach size and

failure times. A sensitivity analysis of breach parameters and times should be performed by running all of the parameter estimates."

Because the breaching process is complex and uncertain, the HEC-RAS manual (2008) suggests that the modeler should try to come up with several estimates of breach parameters and breach sizes, and then put together a matrix of potential breach sizes and times. One example would be use of two different sets of regression equations and one of the breach simulation models to estimate the breach parameters. Hydrologic Engineering Center in Davis California performed several dam breach studies using both the Froelich (1995) and the MLM regression equations, as well as the BREACH model (Fread, 1988). They noticed that all three methods give different answers for the breach dimensions and time. They ran each of these estimates as a separate trial within HEC-RAS to make reliable breach flood analysis.

Therefore, the author tried to use all available empirical breach equations and models, from which conservative breach scenarios are postulated in compliance with the GDC 2 requirement.

6.2 Assumptions Used in Postulating MCR Breach Scenarios

The author used the following assumptions to postulate MCR breach scenarios:

(1) The potential causes of the MCR breach are one or combination of the following three components:

- a piping failure caused by either seepage through the levee or its foundation,
- liquefaction of the foundation soils, or
- levee erosion from rain, flood, or others.

(2) The initial MCR water bevel is 50.8 ft MSL. The initial tailwater condition is dry – There is no rain or flood before the breach. STP determined the initial headwater level by considering the normal maximum MCR operating level of 49 ft MSL plus a margin for wind setup in the reservoir.

(3) A postulated MCR levee breach occurs at the nearest MCR levee section from the units 3 or 4 to maximize the DBF level. The exact placement of the breach section in this study was determined based on the FLO-2D model grid.

(4) Other than the Manning's roughness coefficient (n-value) used in the BREACH model, the conditions and parameter values of the embankment materials are the same as those used by STP.

(5) The author used the STP's BREACH model setup, but levee crest width and tailwater cross-section were corrected to match the field condition.

(6) The breach scenarios include the effects of scour hole, sand chimney and blanket, and cement blocks on the breach process.

By definition, dam is a wall built across a river or stream to stop water from flowing, whereas levee (or dike) is a wall used to manage or prevent water flow into specific land regions. Dams are commonly built on a rigid bottom and side foundation for the stability reasons, however most levees are build on a bare soils. Therefore, the mode of breach and parameters for levees (especially for long levees) could be different from those for dams as:

- Levee breach tends to be wider than dam breach, if the stored water in the reservoir is sufficient in quantity and head
- Levee breach accompanies a deep scour hole
- Tailwater in the breach downstream is characterized by a two-dimensional overland flow whereas dam breach flood is a steep channel flow.

The above factors were considered in this study. As the breach bottom roughness coefficient and scoring hole are critical to determining the beach process and resulting outflows, the following subsections focus on the discussion of the maximum levee breach width and score hole created during the levee breach.

6.2.1 Potential Maximum Levee Breach Width

Some hydrologists assume that there is a physically-acceptable maximum breach width regardless of reservoir-levee conditions. For instance, Von Thun and Gillette (1990) states in their unpublished paper that "the small database of large-dam failures tends to indicate 500 ft as a possible upper bound for breach width." However, this assumption is not true if we consider actual dam failure records. There are numerous reported cases where breach widths for dams and levees exceed far beyond 500 ft. The levee breach of the Martin Cooling Pond in Florida in 1979 is an example where the recorded breach width ended up to 600 ft. STP has been arguing that the MCR breach width of 417 ft is credible considering the historical data, such as the Teton dam failure case. Therefore, the author reviewed the historical breach records to determine the reasonable MCR levee breach width.

Breach Formation Factor

There are many factors that govern the breach process. MacDonald and Langridge- Monopolis (1984) found that the product of the breach outflow volume and the difference in elevation of the peak and base reservoir levels (e.g., $V_w \times h_w$) is a reasonable variable for predicting breach eroded volume and breach width. They called this variable ($V_w \times h_w$) as the breach formation factor. Their finding indicates that the breach process will be continued if there are sufficient water storage and head, or if there is no external factors that interrupt the breach process (e.g., hard rock bottom or abutment, structures, etc). Dams that have very large volumes of water and have long dam crest lengths will continue to erode for a long duration (Gee, 2008). That is, as long as a significant amount of water is flowing through the breach, there will be wider breach width and longer time. This continued breach are also found from the BREACH and FLDWAV simulations (It will be shown later).

Historical Maximum Breach Widths

The author reviewed the USBR dam breach database (Wahl, 1998) to see the maximum breach width that occurred historically. This database has over 108 historical dam breach cases from the world with 410 breach records - There are multiple breach estimation records for a breach in many cases. From this database, the recorded maximum dam breach width of 5800 ft was observed from the Machhu II dam failure in India. Excluding the Machhu II case, the maximum dam breach width is 820 ft, which is followed by 738 ft, 610ft, 551 ft, and so on. These data indicate that the recorded dam breach widths are wider than the STP's postulated MCR breach width of 417 ft.

There are many dam breach database available, however these databases do not include levee breach data or there is no known levee breach database. In the meantime, some reports or papers provide limited levee breach data for selective levee system(s) or region(s). For instance, Nagy (2006) provides 39 levee failure cases in the Eastern Europe Rivers, where

8000 ft is the maximum levee breach width. This maximum is followed by breach width of 1300 ft for the 2nd, 1000 ft for 3rd, and so on.

The URS (2007) report provided 14 historical levee breach cases from the California Delta Levee system, where 1018 ft is the maximum. This maximum is followed by breach width of 950 ft for 2nd, 926 ft for 3rd, and so on.

From these levee breach records, we can conclude that levee breach widths are generally wider than those of dams and that potential levee breach widths could be wider than 1000 ft. This conclusion supports the use of the MLM breach width equation for the MCR breach scenario.

6.2.2 Breach Scour Hole

Scour hole is not a common phenomenon in dam breaches because most dams are constructed on a hard rock foundation. That is why there are not many references that address the scour hole issue in dam breach. However, the scour hole is very common and an important issue in levee breach. The scour hole could increases the capacity (area) of breach outflows significantly and thus for flood levels. Some references address this specific issue (SFWMD, 1980; Zhang and Wang, 2001; Nagy, 2006; etc.).

Scour hole could be occurred right below the levee (Figure 6.2-1) and/or it could be happened at the headwater or tailwater zones or both (SFWMD, 1980; Figure 6.2-2 below). Nagy (2006) pointed out that developing a scour hole (pit) depends on subsurface (foundation) conditions as well as the mechanism of levee destruction. In terms of subsurface condition, scour holes are highly likely if the subsoil below levee is composed of grainy and transitional soils. The width of scour hole is usually smaller than the breach width if foundation is more compacted or less erodible than the levee itself.

SFWMD (1980) reported that the levee breach of the Martine Cooling Pond in Florida in October 1979 created a big scour hole having the dimension of:

- width of 450 ft
- length of 700 ft
- depth of 29 ft from the land surface (or the bottom of the levee), and
- Final levee breach width of 600 ft.

The scour hole for the Martine Cooling Pont has a U shape across the flow direction, having a wide top width but narrow bottom width (about 190 ft wide and 400 ft long). The maximum depth was appeared at slight downstream from the levee.

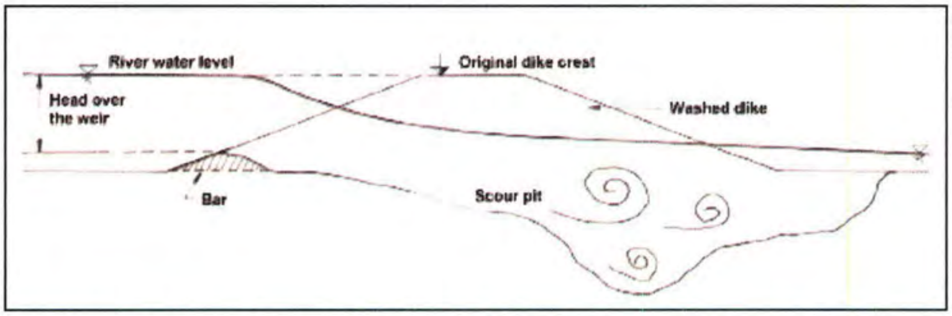


Figure 6.2-1. Typical cross section of a levee failure with scour hole (pit) (after Nagy, 2006).

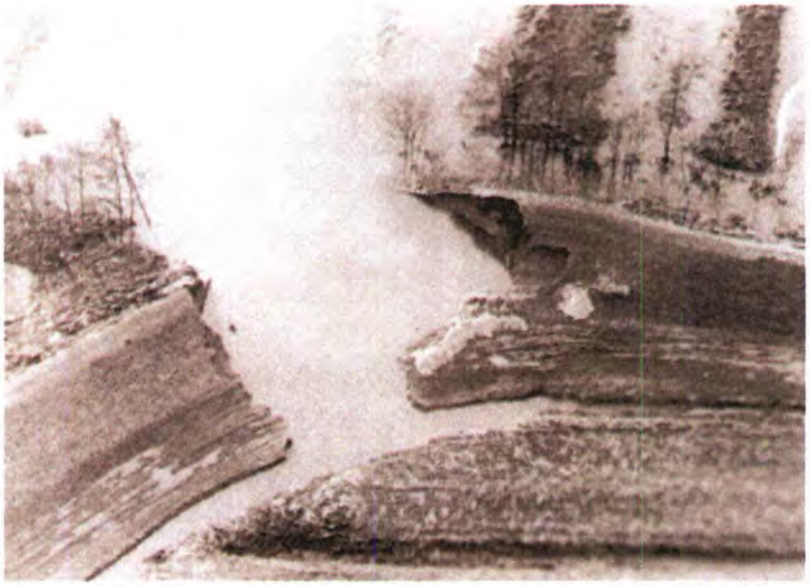


Figure 6.2-2. Potograph of Elba River bank breach in 1976 (Nagy, 2006)

Zhang and Wang (2001) simulated the depths of scour hole from levee breach using a numerical model. The model is based on a solution of 2-dimentional shallow water and suspended sediment equations. They estimated that, under the breach width of 328 ft and the median grain side of 0.3 mm, the maximum scour hole depth from the surface is estimated to be about 20 feet. Their study indicates that the maximum scour hole occurs at the tailwater portion (e.g., after the toe) of levee or about 100 to 165 meters downstream from the invert of breach. They found that the hole occurs at the early stage of breach (1.7~10 minutes).

Based on the result of Zhang and Wang (2001), STP postulated a scour hole from the MCR breach at the toe of levee. This hole has width of 380 ft, length of 203 ft, depth of 20 ft from the surface elevation 29 ft MSL. They used the hole in estimating the volume of eroded materials to evaluate the effects of sedimentation deposition at the site on the DBF flood level. However, STP assumed no scour hole underneath the MCR levee thus no impacts of the hole on the MCR breach process and outflows. They justified that the road right after the levee prevent scour hole below the levee without proper justificaiton. STP's ignoring the scour hole in the MCR breach results in significant underestmation of the DBF level as my analysis shown later. This is a significant drawback in terms of a site safety analysis.

The author considered the following three cases to consider the scour hole effects in MCR breach:

- Case 1: No scour hole under the levee as in the case of the STP scenario
- Case 2: Extend the scour hole to the levee below, with an average hole depth of 10 ft (i.e., 0 ft on the invert and 20 ft at the toe of levee)
- Case 3: Extend the scour hole to the levee below, with an average hole depth of 20 ft (i.e., an average of 20 ft depths at both invert and toe of levee)

The pictorial descriptions of these cases are shown in the following figure. This study considers the above three cases in the breach flood analysis later.

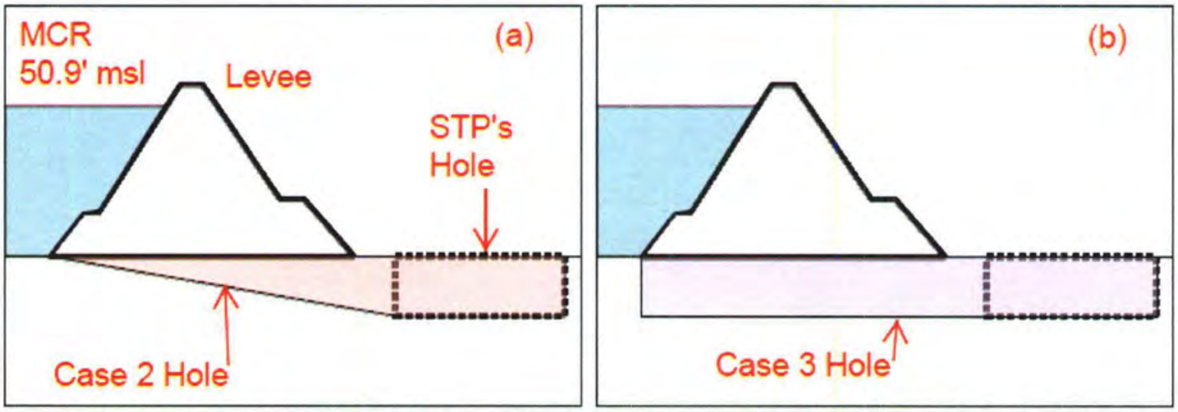


Figure 6.2-3. Schematic plots of MCR breach scenarios with different scour holes: (a) case 2 with 10 ft hole depth, (b) case 3 with 20 ft hole depth (not to scale).

6.3 Breach Parameter Estimation Using Empirical Equations

6.3.1 Dam Breach Analysis

In general, dam breach analysis includes dam breach simulation and downstream flood routing. The analysis uses either empirical regression equations or numerical models, or both. Because there is no model that can perform the two simulations together, hydrologists use a multi-step approach. For instance, breach parameters are estimated using empirical equations and them provided the parameters as input to breach outflow and flood routing models. The breach parameters include width (B) and time (Tf) of breach, rate (Qp) and time (Tp) of peak outflow, and breach outflow hydrograph (Q(t)) (refer to Figure 6.3-1). Several methods are available for estimating breach parameters (Wahl (1998):

- Simple regression equations to predict breach width and time
- Simple regression equations to predict peak outflows
- Physically-based breach simulation models, such as BREACH, FLDWAV, HEC-RAS, FLO-2D, etc.

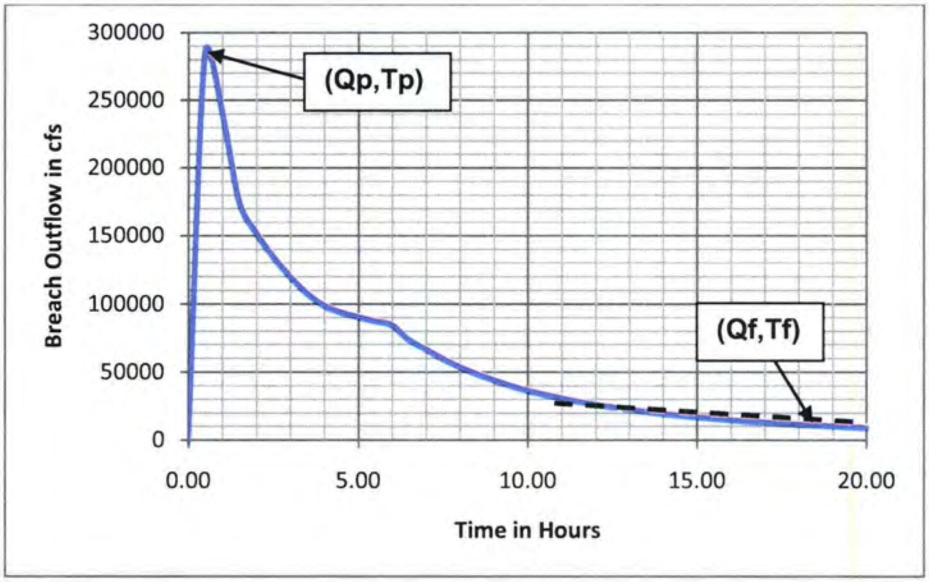


Figure 6.3-1 An example of breach outflow hydrograph simulated by the FLDWAV model for the breach width of 500 ft and the time to peak of 0.5 hours.

Regression equations estimates breach parameters as functions of one of more dam and reservoir parameters, such as storage volume (V_w), depth of water of depth of breach (h_w), etc. The equations are based on regression analyses of data collected from actual dam failures. Several federal agencies, such as USBR, provide the database of historical dam failures.

However, these databases are relatively lacking in data from failure of large dams, with about 75 percent of the case having a dam height less than 50 ft (Wahl, 1998). Further, they do not include levee failure records.

Chauhan et al. (2004) concluded, based on a simulation of DAMBRK (Fred,1984) and an empirical dam breach equation, that the use of breach parameters estimated from empirical equations as inputs to physical breach models such as DAMBRK typically results in significant overestimation of peak breach flows. However, their case study uses very limited breach conditions – It could end up different conclusion if one use different equations, models, and breach conditions. In fact, many dam breach simulation studies (e.g., Gee, 2008; Gee, 2010; Wahl, 1997, etc.) show that physical models give either higher or lower peak flows depending on site conditions.

As my study demonstrated later, physical models, like BREACH or FLDWAV, tend to produce overestimation of peak flow and time. It is my opinion that the breach parameter estimates using empirical equations are more credible than those by physical model because (1) empirical equations are based on real data, (2) validations of physical models are usually done with limited number of historical records or no recorded at all for future breach prediction, (3) breach parameter estimates using physical models are widely vary depending on model inputs which are by and large uncertain.

Breach side slope along the breach width and depth is a factor that specifies the shape of breach opening. However accurate predicting the breach side slope angle is of secondary importance to predicting the width and depth (Wahl, 1998). Therefore, this factor is not considered in the MCR breach analysis. As the side slope is insensitive to the breach outflow, this study used a fixed breach side slope angle of 45^{o} (1h:1v slope) in BREACH modeling.

Most empirical breach time equations are for the final breach time (Tf). However, the breach time that is critical in flood hazard analysis is not Tf but the time to peak flow (Tp). Breach time estimated by physical breach models (e.g., BREACH, FLDWAVE, etc.) tends to be longer than those using empirical breach time equations. This long breach time may result in less conservative breach flood hazard.

Physically-based breach models are available to aid in the prediction of breach parameters. There are many reported physical breach models (Wahl, et al., 2008; Gee, 2008, etc.), including NWS-BREACH, SIMBA, HR-BREACH, FIREBIRD-BREACH, etc. Although none are widely used, the most notable is the National Weather Service BREACH models (Fread, 1988: NWS-BREACH or BREACH hereafter). Physical models are intended to simulate the hydraulic and erosion processes associated with flow from overtopping or piping failures of a dam and obtain breach parameters and breach outflow hydrographs.

Primary weakness of these physical breach models is the fact that they do not adequately model the erosion process that dominates the breaching of cohesive-soil embankments (e.g., Hahn et al. 2000). This is why many hydrologists recommend using multiple methods (both

empirical equations and physical models) in a dam breach analysis. However, both STP and PNNL used only the selective empirical equations or modeling. Instead, the author tried to use all available empirical equations and multiple models in a comprehensive manner.

6.3.2 Breach Parameter Estimation Using Empirical Equations

The listed below summarizes the empirical regression equations for estimating width and time of breach and peak breach outflow (Wahl, 1997):

- U.S. Bureau of Reclamation (1988)
  - Average breach width (m), $B_{avg} = 3\ h_w$
  - Failure time (hr), $Tf = 0.011\ B_{avg}$
  - Peak flow (m^3/s), $Qp = 19.1\ h_w^{1.85}$

- MacDonald and Langridge-Monopolis (1984)
  - Volume of eroded material (m^3), $B_{avg} = V_{er}/A = 0.0261\ (V_w\ h_w)^{0.769}/A$
  - Failure time (hr), $Tf = 0.0179 V_{er}^{0.364}$ for bounding, or $Tf = 0.0704\ (V_{er})^{0.2874}$ for best fit
  - Peak flow (m^3/s), $Qp = 3.85\ (V_w\ h_w)^{0.411}$ for bounding

- Von Thun and Gillette (1990)
  - Average Breach width, $B_{avg} = 2.5 h_w + C_b$
  - Failure time (hr), $Tf = B_{avg}/(4h_w+61)$ for highly erodible dam materials
  - Peak flow (m^3/s), not available

- Froehlich (1995a, b)
  - Average breach width (m), $B_{avg} = 0.1803\ K_o\ V_w^{0.32}\ h_b^{0.19}$
  - Failure time (hr), $Tf = 0.00254\ V_w^{0.53}\ h_b^{-0.9}$
  - Peak flow (m^3/s), $Qp = 0.607\ V_w^{0.295}\ h_w^{1.24}$

where

h_w = depth of water above breach invert (m)

V_w = volume of water stored (m^3)

C_b = offset factor which is a function of reservoir size:

K_o = overtopping multiplier (1.4 for overtopping, 1.0 for piping)

h_b = height of breach (m), and

A = breach cross section area (m^2).

The MLM(1984) paper presents a bound failure time equation but not a best fit failure time equation. Therefore, the author obtained the above best-fitted breach time equation using 14 earth-fill dam failure data in the MLM's paper. The figure below shows the regression analysis where the historical data of erosion volume and time were log-transformed to fit a linear line as in the case of MLM's breach erosion volume equation. The linear regression equation was back-transformed to get an exponential breach time equation as shown above. The MLM best-fitted breach time equation is used in this study for consistency with other equations.

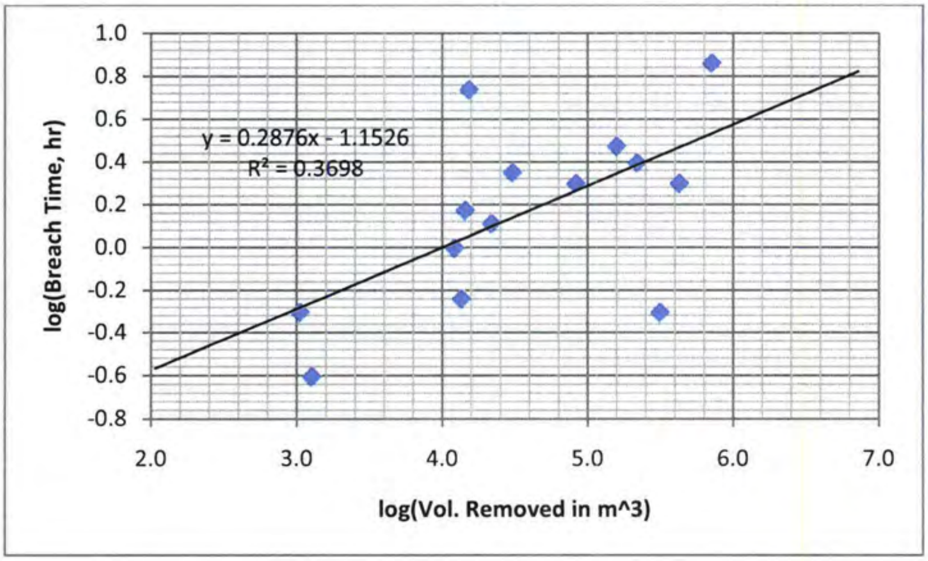


Figure 6.3-2. The best-fitted MLM breach time regression line that was obtained with the dam breach data provided by the MLM paper (1984).

The author used an initial MCR level before breach of 50.9 ft MSL and the bottom levee elevation of 29 ft MSL. The corresponding reservoir head (h_w) and storage volume (V_w) are 21.9 ft and 6.653×10^9 ft^3. Table 6.3-1 lists estimated breach parameters using the empirical equations above. The MLM breach width estimate in Table 6.3-1 is based on the scenario of no scour hole. The Table 6.3-2 compares the MLM breach widths for different scour hole depths. Table 6.6-3 lists peak breach outflow rates using 10 different empirical equations which are presented in Wahl's paper (1997). Some of the peak flow equations in Wahl (1997) were excluded in Table 6.3-2 because they are a bounding equation. However the envelop USBR peak flow equation is included because it is used in Table 6.3-1.

Table 6.3-1. MCR breach parameters estimated by four sets of empirical equations.

Equation	Breach Width (ft)	Failure Time (hr)	Peak Flow (cfs)
Bureau of Reclamation	66	0.66	22,607
MacDonald and Langridge-M	1736	2.54	217,000
Von Thun and Gillette	235	2.68	n.a. (1)
Froehlich	417	7.98	60,078

Note: (1) n.a. = not available.

(2) Cross section area of 5281 ft^2 at the breach bottom level of 29 ft MSL was used for the MLM breach width equation here.

Table 6.3-2. MLM breach width estimates using different scour hole scenarios

Scenario	Scour Hole Depth (ft)	Levee Erosion Volume (ft3)	MLLM Breach Width (ft)
Case 1	0 (no scour hole)	5281	1736
Case 2	10 ft	8754	1047
Case 3	20 ft	12309	745

Table 6.3-3. Breach preak flow equations and their estimated values for MCR.

Author	Equation	Qp(cfs)
Kirkpatrick (1977)	$Qp=1.268(h_w+0.3)^{2.5}$	5,754
SCS (1981)	$Qp=16.6(h_w)1^{.85}$	19,648
USBR (1982)-Envelop	$Qp=19.1h_w1^{0.85}$	22,607
Singh & Snorrason-1 (1984)	$Qp=13.4h_d^{1.89}$	46,105
Singh & Snorrason-2 (1984)	$Qp=1.776\ S^{0.47}$	458,501
Hagen (1982)	$Qp=0.54(S.h_d)^{0.5}$	826,102
MLM (1984)	$Qp=1.154(V_w{*}h_w)^{0.412}$	**217,247**
Costa (1985)	$Qp=0.981{*}(s{*}h_d)^{0.42}$	271,919
Evans (1986)	$Qp=0.72{*}V_w^{0.53}$	578,760
Freohlich (1997)	$Qp=0.607{*}V_w^{\ 0.295}\ h_w^{1.24}$	60,078
Average of 10 estimates		**251.000**
Average of last 5 estimates		**336,000**

Where, Qp = peak breach outflow in (m^3/s)
S = Reservoir storage (m^3), and
h_d = height of dam (m).

Based on the result of the above estimations, the following discussions were made:

- Both the Bureau of Reclamation and Von Thun and Gillette methods tend to underestimate breach width and peak flow rate because they are estimated as a function of storage or head only.

- USBR's dam breach manual (Wahl, 1998) states that the Froehlich's equation appears to be the best predictor for the breach widths less than 164 ft because the breach width records used to developed for the Froehlich equation is small. In other words, the Froehlich's breach width equation is not applicable to the MCR breach case.

- Therefore, this study considers the MLM method only. The MLML breach width range from 745 ft to 1736 ft with and without scour hole. The MLM estimates are comparable to the results of BREACH and FLDWAV simulation as shown later.

- Also the MLM peak flow estimate of 217,000 cfs is roughly consistent with an average (251,000) of 10 peak flow estimates shown below. It should be noted that the first five peak flows are underestimated because they ignore the MCR storage volume. Excluding these five, an average of peak flow is 336, 000 cfs.

The above discussions lead to a conclusion that the breach parameters estimated by STP (B=417 ft, Tf=1.7 hrs, Qp=130,000 cfs) are underestimated, resulting in an underestimation of the DBF level. This conclusion will be further validated using the simulation of numerical breach and flow models later. However, the STP' and PNNL's dam breach analyses ignored or omitted the above empirical equations-derived information partly or entirely without proper justification.

6.4 Breach Parameter Simulation Using the BREACH Model

The author used the NWS-BREACH model to supplement the breach parameter estimates by empirical equations. The BREACH model is probably the best known physically-based numerical breach model. BREACH model predicts breach parameters (size, shape, time of formulation) and breach outflow hydrograph emanating from a breached earth dam (Fread, 1993 & 2000). This model simulates coupling of the specific erosion processes and the flow process by breach. The BREACH model also accounts for the dynamic effects of the flow within the upstream reservoir using a simple mass-balance type reservoir routing and the tailwater effects on the flow through the breach. Because the BREACH model use only a limited immediate reach of the tailwater channel, additional flow routing model is needed for simulating the flood hazard far downstream.

The BREACH model uses the the Meyer-Peter & Muller (MPM) sedimentation equation for dam breach process and the 1-D Manning's uniform flow equation for the breach outflow. These equations are function of the Manning's roughness coefficient which has been known as one of the most critical parameters in the BREACH model (refer to Figure 6.4-1). STP uses n-value of 0.005 in their BREACH modeling, which seems too small for sediment-oriented breach flows. This n-value of 0.05 results in the peak breach outflow of 83,000 cfs which is also small compared to the estimates by other methods.

PNNL (2011) selected n-value of 0,075 without justification. PNNL performed a sensitivity analysis, but part of their sensitivity analysis is erroneous due to a faulty BREACH setup. Therefore, the author investigated the proper n-value for the MCR breach, followed by a supplemental sensitivity analysis of the MCR BREACH model.

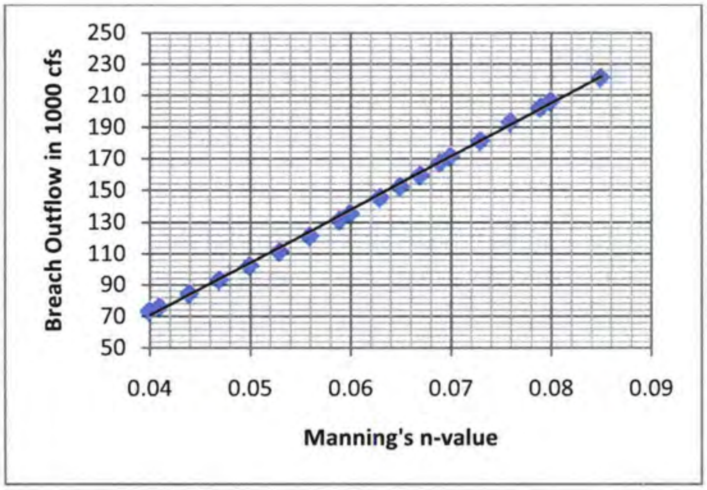


Figure 6.4-1. The relation between Manning's n-value and breach peak outflow obtained from BREACH simulations (without scour hole effects).

6.4.1 Manning's Roughness Coefficients

There are many referenced n-values applicable to channel and overland flows. However, there is no specific guidance that directs to select n-value for dam breach simulations. STP's FSAR states that, for BREACH modeling, an n-value within the range from approximately 0.025 to 0.05 was considered reasonable, while a higher n-value of 0.08 is still considered within the range of feasibility. The n-value of 0.075 is used in SER. The BREACH manual proposed to use the Straler's method to estimate n-value. However, the estimated n-value with this equation is too small because the size of the embankment material (silty clay) is small (0.001 mm).

Chow (1969) states that n-values for sediment laden flow is higher than without it. Since n-value in the BREACH model is critical in breach process, this subsection estimate the n-value for the MCR breach considering the embankment materials, sand chimney, and cement blocks. The author used the Chow's method to estimate the n-value for the MCR breach. The estimated value was compared with other literature values.

<u>Chow's Method to Estimate n-value</u>

This report estimated Manning's n-value using the method presented by Chow (1959; Arcement and Schneider, 1984). Table 5-5 in Chow (1959) provides a procedure for estimating specific n-value considering the effects of bed materials and shape of channel as:

$$n=(n_b+n_1+n_2+n_3+n_4)m \qquad \text{(Equ. 6.4-1)}$$

where
- n_b is the base n value
- n_1 is the irregularly factor
- n_2 is the channel cross section factor
- n_3 is the obstruction factor
- n_4 is the vegetation factor, and
- m is the meandering factor.

The parameters in equation (6.4-1) are estimated based on the above two references as following:

- n_b is 0.02 for earth bed materials which consisted of sand with the median diameter of 0.4 mm.
- n_1 is 0.015 for an average of moderate and severe irregular channels caused by badly eroded or sloughed sides of canals. The maximum n_1 for severe irregularity is 0.02.
- n_2 ranges from 0.01 to 0.015 for frequently alternating of channel cross section due to the constriction at the breach invert area and wide expansion at the breach outlet. The rapid contraction and expansion create highly turbulent breach flow.

- n_3 ranges from 0.02 to 0.03 for appreciable obstruction caused by the littering of clay cement blocks that fall and cover about 40% of the breach surface area during the breach process.
- n_4 is 0.005 for small vegetation. The maximum n_4 is 0.01.
- m is 1 for minor (minimum) meandering.

Using the above, the low and high n-values are computed as 0.07 and 0.085, respectively, with an average n-value of 0.0775. These estimated values are compared with other tabulated and referenced n-values as the table below.

Table 6.4-1. Comparison of the Manning's roughness coefficients (n-values) for difference sources.

Source	n-value		
	Minimum	Medium	Maximum
FSAR	0.025	0.05 (used)	0.08
SER	0.025	0.05	0.075 (used)
Chow Method (this study)	0.07	0.0775	0.085
Table 5-6 in Chow (1959) – major stream (B>100 ft) with irregular and rough section	0.035	-	0.10
FLO-2D MANUAL – overland flow	-	0.07	-
Estimated based on the MLM peak flow of 217,000 cfs	-	0.085	-

In summary, n-values for the breach process could range from 0.025 to 1.0. The estimated n-value in this study for the MCR breach considering the breach condition, sand chimney, and cement blocks is 0.0775, but the BREACH model in this report uses an n-value of 0.075 in line with the SER.

6.4.2 BREACH Sensitivity Analysis

The BREACH model uses the following input parameters:

- Dam materials: grain size, porosity, unit weight, n-value, frictional angle, cohesive strength, and D90/D30 ratio for both inner and outer materials
- Dam properties: (1) for clay materials, PI index, critical shear stress, and critical stress coefficient; (2) slope of upstream and downstream faces, and slope of core; (3) for grass cover, length, condition, max erosion velocity, max sediment concentration
- Breach parameters: ratio of initial width to depth, width of dam, tailwater slope, particle size (D50), ration of D90 and D30, max bottom width, and max top width

- Elevation: initial reservoir level, top and bottom elevations of dam, initial piping elevation, and spillway crest.
- Others: tailwater cross section, simulation time step, and output control.

PNNL performed a BREACH sensitivity analysis to determine the sensitivity of inputs on the MCR levee breach process and outflows (PNNL, 2011: refer to the PNNL's calpack). Based on the sensitivity analysis, PNNL concluded that:

- The BREACH model predictions are very sensitive to n-value.
- The BREACH model predictions are slightly sensitive to internal frictional angle.
- The breach predictions for peak flow and time and breach width are fairly insensitive to initial piping elevation, cohesive strength, frictional angle,
- The model predictions are very sensitive to n-value.

However, PNNL made erroneous sensitivity conclusions for (1) length of levee as not at all sensitive; and (2) tailwater cross section as slightly sensitive. Therefore, the author performed an additional sensitivity analysis for n-value, levee length, tailwater cross section, and scour hole as described below.

<u>Formulate a Base Run Scenario</u>

To start with, the author postulated the following four MCR breach scenarios for which the result of BREACH simulations is summarized in the table below:

- FSAR scenario: {N=0.05,C=300 lb/ft2, phi=20^{O}}
- SER scenario: {N=0.075,C=200 lb/ft2, phi=15^{O}}, but all other parameters are the same as those for the FSAR scenario.
- RUN1: A corrected SER scenario with tailwater cross section and max levee length of 3000 ft. Note: MCR BREACH gives a runtime error if width is greater than 3000 ft.
- RUN2: RUN 1 with a 10-ft scour hole.

Table 6.4-2. Comparison of BREACH simulations with different MCR breach scenarios..

Run	Condition	Peak				Final Breach$^{(2)}$			
		Qp	Tp	Breach Width		Qf	Tf	Breach Width	
		kcfs	hour	Top, ft	Bot., ft	kcfs	hour	Top, ft	Bot.,ft
FSAR	n=0.05	83	6.25	412	361	26	22.51	498	446
SER	n=0.076	128	1.99	574	463	23	20.51	747	637
RUN1	Corrected	194	3.29	989	878	24	17.39	1259	1149
RUN2	Scour hole	269	2.12	703	562	26	21.64	861	720

Notes:

- Kcfs is the 1000 cubic feet per second.
- RUN 2 is the base run for the flowing sensitivity analysis.

The tables above and below provide peak breach parameters as well as those at the end of the breach process in order to compare those using empirical equations. The final breach time is almost infinite, but they were determined using the criteria of (1) the difference between the headwater level and tailwater level (e.g., HY-TWD parameter from BREACH output) become less than 1 ft or (2) breach outflow is less than 27,000 cfs which is about 10% of RUN 2 peak outflow. The peak parameter values for the "SER run" are from the PNNL calpack – These are slightly different from the simulation of this study. From this simulation, the following discussions are made:

- The result of this simulation indicates that the corrected BREACH model produces higher peak outflow, and even higher if the effects of scour hole are considered.

- The breach time simulated by the BREACH model tends to be longer than the ones by empirical equations. However, the tail of the outflow is not significant in determining the DBF level, thus ignored.

- The breach process continues after the peak flow, but the rate of the breach process is relatively slow compared to that before reaching the peak outflow.

<u>BREACH Sensitivity Runs</u>

Table 6.4-3. Sensitivity of Manning's n-value to BREACH model results.

Run ID	n-value	Peak				Final Breach			
		Qp	Tp	Breach Width		Qf	Tf	Breach Width	
		kcfs	hour	Top, ft	Bot., ft	Kcfs	hour	Top, ft	Bot.,ft
R11	0.10	323	1.06	820	679	60	9.68	1018	877
R12	0.08	280	1.52	723	583	46	12.24	892	751
BASE	0.075	269	2.12	703	562	26	21.64	861	720
R13	0.06	224	2.96	623	483	31	16.5	745	604
R14	0.04	139	6.20	481	340	32	19.4	557	416

Table 6.4-4. Sensitivity of scour hole size to BREACH model results.

Run ID	Hole Depth	Peak				Final Breach			
		Qp	Tp	Breach Width		Qf	Tf	Breach Width	
	Ft	kcfs	hour	Top, ft	Bot., ft	kcfs	hour	Top, ft	Bot.,ft
R21	0	195	3.63	1029	918	26	16.40	1676	1565
R22	5	252	3.04	875	749	26	18.93	1107	981
BASE	10	269	2.12	703	562	26	21.64	861	720
R23	15	271	1.80	595	439	26	24.65	716	560
R24	20	267	1.64	518	347	26	14.8	620	450

Notes: R21 and RUN1 in Table 6.4-2 are identical.

Table 6.4-5. Sensitivity of tailwater cross section to BREACH model results.

Run ID	Tailwater Width	Peak				Final Breach			
		Qp	Tp	Breach Width		Qf	Tf	Breach Width	
	Ft	kcfs	hour	Top, ft	Bot., ft	kcfs	hour	Top, ft	Bot.,ft
BASE	3000	269	2.12	703	562	26	21.64	861	720
R31	2000	244	0.77	611	470	24	24.42	849	709
R32	1500	219	0.78	570	430	24	24.92	798	658
R33	1200	204	0.98	570	429	25	33.39	781	641
R34	1000	195	0.79	526	386	26	25.14	771	630
R35	800	159	1.10	489	349	23	29.33	679	540
R36	600	122	0.80	388	248	25	29.84	567	426

Note: (1) On these sensitivity runs, levee length was fixed to 3000 ft.
(2) The tailwater section width greater than 3000 ft give a runtime error on the MCR BREACH simulation.

Table 6.4-6. Sensitivity of levee length to BREACH model results.

Run ID	Dyke Length	Peak				Final Breach			
		Qp	Tp	Breach Width		Qf	Tf	Breach Width	
	Ft	kcfs	hour	Top, ft	Bot., ft	kcfs	hour	Top, ft	Bot.,ft
BASE	3000	269	2.12	703	562	26	21.64	861	720
R461	600	265	0.62	600	488	26	15.39	600	488
R462	500	220	0.47	500	388	25	16.74	500	388
R47	400	167	0.39	400	288	25	22.04	400	288

Note: On these sensitivity runs, width of tailwater cross section was fixed to 3000 ft.

The above four tables summarize the result of sensitivity analyses for n-value, scour hole depth, width of the bottom tailwater section, and levee length, respectively. From this sensitivity runs, the following discussions are made:

- The BREACH model predictions are very sensitive to n-value. This was also noted in the PNNL's calpack and Figure 6.4-1.
- The BREACH model predictions are very sensitive to the scour hole depth. Ehe effective of scour hole is nonlinear - The peak outflow rates increase with increasing hole depth. The flow rates are peaked (271 kcfs) at the hole depth of 15 feet, then decrease thereafter. It seems that this peak flow is the probable maximum MCR breach outflow, if scour hole is considered in the breach process.
- Both peak time and breach width is inversely proportional to the hole depth. The deeper the scour hole is, the more the initial breach outflows is. However, scour hole reduces breach width. In conclusion, considering the effects of breach scour hole induce high peak breach outflows and more severe flood hazard than without considering it.

- In contrast to the PNNL's conclusions, the sensitivity analysis by this study demonstrates that BREACH predictions are very sensitive to both levee length and the width of tailwater cross section. These are not variable. However, use of correct and realistic values for these two inputs is needed in BREACH modeling.

6.5 Breach Outflow Simulation Using the FLDWAV Model

The FLDWAV model (Fread, 1993) is a generalized unsteady-flow model for open channels. It replaces the DAMBRK, combining its dam breach simulation capabilities and providing new hydraulic simulation procedures. FLDWAV can simulate dam failures caused by either overtopping or piping. The program can also represent the failure of two or more dams located sequentially on a river. The program is based on the complete equations for unsteady open-channel flow (St. Venant equations). This model has options to simulate various external and internal boundary conditions and hydrologic structures.

Fread (2000) states that "BREACH should be used judiciously and with caution - BREACH model is intended to be an auxiliary method for determining the breach parameters and should be used in conjunction with statistical and range of magnitude data from historical breaches." Therefore, the BREACH model has not been directly incorporated into FLDWAV. However, FLDWAV can easily simulate breach outflow hydrographs using breach parameters (width and time) as input. This feature gives an advantage of bypassing the burden of identifying the BREACH input parameters, especially the Manning's n-value. After setting up the FLDWAV model for the MCR breach case, the author performed a comprehensive sensitivity analysis to understand the behavior of the MCR FLDWAV model.

FLDWAV Model Set-up

The MCR FLDWAV model was set up with available data from FSAR and RAI responses. The MCR FLDWAV model consists with three hydrologic components: MCR reservoir routing, MCR levee breach, and tailwater routing. All of these hydrologic components are simulated in a 1-dimentional way because FLDWAV does not have capability to simulate 2-D flows. The MCR FLDWAV model uses five channel cross sections: The first cross section is for MCR reservoir with levee, and remaining four are for tailwater reaches.

First, MCR reservoir is used as an upstream boundary condition. Relevant reservoir input to FLDWAV include MCR stage-surface area rating curve, starting and bottom reservoir elevations, and upstream inflow (discharge) boundary condition. There is no actual upstream inflow into the MCR during the breach, but small inflow values were input to the model to provide a stability of the channel routing - FLDWAV gives a runtime error without this fictional input.

Second, levee breach input include elevations of top and bottom of levee, weir coefficient, breach time, breach width, initiating reservoir level, reservoir bottom level, levee crest length, initial piping level, and breach side slope (1h:1v). These values are same as those for the MCR BREACH model. Again, width and time of breach is the most critical parameters in determining breach outflows.

Third, tailwater flow condition was assumed as a 1-D flow on a wide channel (4 miles width) to mimic the 2-D overland flow condition at the power block area. This tailwater condition is more realistic than that of the MCR BREACH model that uses only one tailwater cross section with a

width of 3000 ft. Tailwater input include number of cross sections, Manning's n-value for channel (specified from 0.03 to ~0.08, but actual n-values are calculated by the model internally depending on the condition and depth of flows), length and slope of each channel reach, and cross section geometry.

After setting up the MCR FLDWAV model, it was tested for the STP's breach scenario (e.g., B=415 ft, Tf=1.7 hours). The test run provide the same peak outflow (128,000 cfs), validating that the model was set up correctly. The author performed a comprehensive sensitivity analysis in order to understand the behavior of the MCR FLDWAV model.

FLDWAVE Sensitivity Runs

The author simulated the MCR FLDWAV model with different widths and times of breach. The upper bound of the breach parameters are determined conservatively (5000 ft width and 3 hours breach time). For discrete values of breach parameters, 42 FLDWAV runs were performed. The following two tables list simulated peak outflows and times to peak. They were also plotted in the following two figures.

Tp\B_avg	500 ft	750 ft	1000 ft	1500 ft	2000 ft	3000 ft	5000 ft
0.5 hr	285	415	539	735	830	933	1077
1.0 hr	277	391	466	535	590	670	779
1.5 hr	265	331	369	425	469	534	635
2.0 hr	241	280	309	357	398	454	523
2.5 hr	210	244	271	312	348	397	475
3.0 hr	189	219	241	280	306	361	426

Table 6.5-1. Peak breach outflows (in 1000 cfs) simulated by the MCR FLDWAV model with different breach widths and times.

Tp\B_avg	500 ft	750 ft	1000 ft	1500 ft	2000 ft	3000 ft	5000 ft
0.5 hr	0.5	0.5	0.5	0.5	0.5	0.425	0.35
1.0 hr	1.0	1.0	1.0	0.9	0.8	0.7	0.6
1.5 hr	1.5	1.5	1.35	1.20	1.05	0.9	0.75
2.0 hr	2.0	1.8	1.6	1.4	1.3	1.1	0.9
2.5 hr	2.375	2.125	1.875	1.625	1.5	1.375	1.125
3.0 hr	2.7	2.4	2.1	1.95	1.65	1.5	1.2

Table 6.5-2. Peak breach time (in hour) simulated by the MCR FLDWAV model with different breach widths and times.

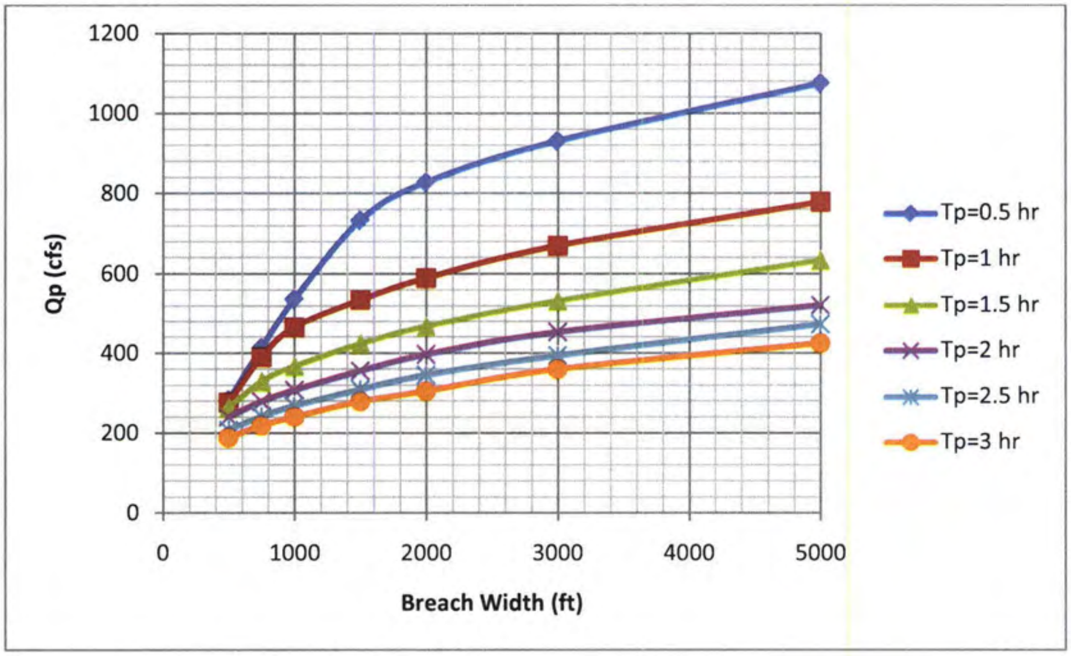


Figure 6.5-1. Breach peak outflows estimated by FLDWAV with different breach widths and times.

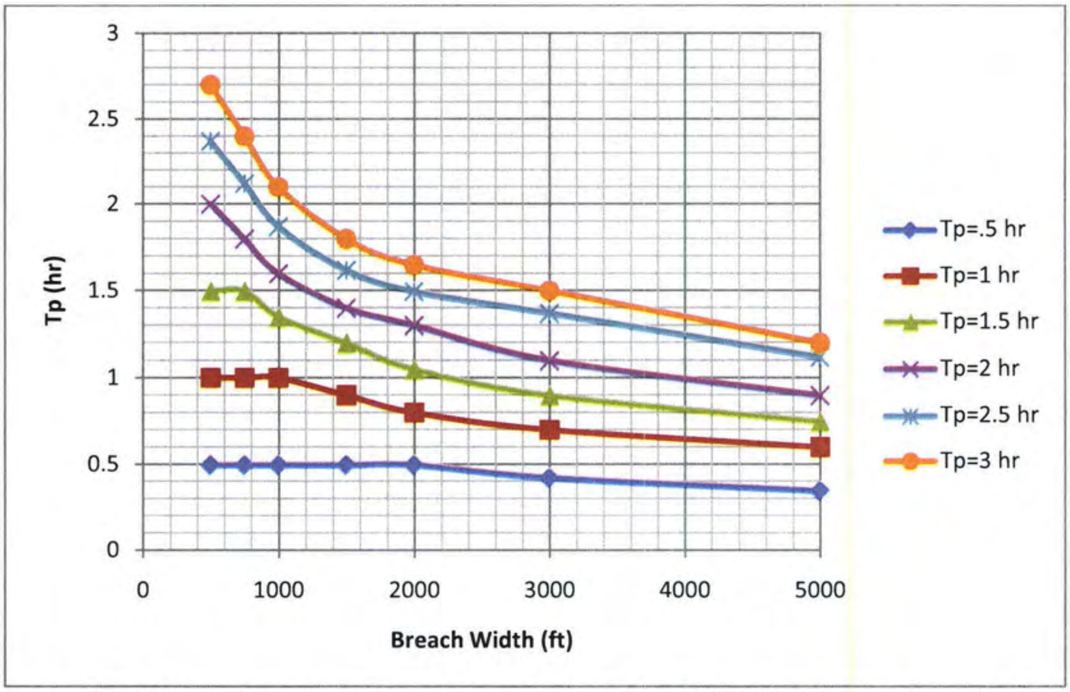


Figure 6.5-2. Breach peak outflows estimated by FLDWAV with different breach widths and times.

From these sensitivity runs, the following discussions were made:

- The peak outflow is proportional to breach width, but the rate of increase is nonlinear. The peak outflow increases shapely for the breach width less than 2000 ft, but the rate of increase is reduced thereafter.

- The pink color cells in Table 6.5-2 are the case where FLDWAVE-simulated breach peak time is less than the corresponding peak time input. The reduction is more significant for wider breach widths. The reduction indicates that peak time in FLDWAVE model input is unrealistically small so that the FLDWAVE model adjusts the outflow and time to peak based on the breach condition and geometry (breach width and others).

- The two figures above are useful in determining the peak outflow and time given the breach width and time obtained from empirical equation. In cases where peak outflow and time are estimated independently, it is necessary to filter and adjust the parameters using the FLDWAV runs. Figure 6.5-1 and 6.5-2 are useful in adjusting the peak outflow and time as shown in Subsection 6.6.6.

6.6 Flood Routing Using the FLO-2D Model

In general, dam breach flood analysis in a river valley or channel has been performed with 1-dimensional (1-D) flood routing models such as HEC-RAS or FLDWAV. In such cases, dam breach flood wave is confined laterally by natural abutments and well-defined downstream river channels, therefore an implicit assumption of velocity vectors normal to the river cross-sections is reasonable for dam breach analysis. However, levee breach flood routing for a large off-stream reservoir constructed on flat ground must be done with a 2-dimentional (2-D) simulation with velocities and flows dispersing in any direction. The MCR levee breach flood at the power block area should be handled by a 2-D hydrodynamic model. The main difference between 1-D and 2-D simulations is that, channel cross sections for 1-D simulation are replaced with a continuous topographic surface for 2-D simulation.

The MCR levee breach flood analysis in this study is based on a combined approach of 1-D simulation of breach outflows and 2-D routing of overland flow after the breach. Because the RMA2 model that was used by STP is not available to the NRC staff, the author used the FLO-2D model. FLO-2D is a dynamic routing model that simulates channel flow, unconfined overland flow and street flow. It can simulate a flood over complex topography and roughness while reporting on volume conservation which is the key to accurate flood routing. FLO-2D uses a full dynamic wave momentum equation and a central finite difference routing scheme with eight potential flow directions over a system of square grid elements. The equations of motion in the FLO-2D finite difference model grid are defined as a quasi two-dimensional model, which are solved by computing the average flow velocity across a grid boundary one direction at a time. FLO-2D does not distinguish between subcritical and supercritical flow because the momentum equation used in the model has no restrictions when computing the transition between the flow regimes. Because of this advantageous feature, the FLO-2D model has been widely used in simulating levee breach floods.

6.6.1 FLO-2D Model Set-up

The author set up the MCR FLO-2D model to simulate the MCR breach induced flood at the power block area. The FLO-2D model has options to simulate various types of flows, including overland flow, channel flow, flood plain, levee, dam and levee breach, hydraulic structures, street flow, rain-runoff-infiltration process, sediment transport, mud and debris flow, and others. However, the MCR FLO-2D model utilizes only the overland flow option and limited options for levee and levee breach because it uses the breach outflow hydrograph simulated by the FLDWAV model as upstream boundary inflow. All other options were omitted, making the MCR FLO-2D model very simple and effective in running. The input to the MCR FLO-2D model are as following:

- Static data: surface elevation and n-value in space, drainage features (location, width, depth), features of ponds and reservoirs (extend, elevations, levee information), features of structures and roads (location, elevation, etc).

- Boundary conditions: upstream boundary inflow hydrograph, location of outflow boundary nodes – they are not sensitive to the DBF level.

- Levee information: Location, top and bottom elevation

- Breach information: location (nodal points) and width of breach

- Numerical tolerance input: surface detention factor, allowable percent change in flow depth, and dynamic wave stability factor.

The boundary of the MCR FLO-2D model was determined by a trial and error so that the model area is small enough to run efficiently but is large enough to avoid the effects of the downstream boundary on the estimated DBF level (Figure 6.6-1). The model domain in this study is larger than that of the STP's RMA2 model. The model domain is divided into uniform, square grid nodes (200 ft resolution). It has a total of 29948 nodes. The nodal points for structures and MCR levee were assigned in the model.

Breach failure location was identified as the closed levee section from the proposed units 3 and 4. The model was set up so that the location of the breach section is used as an inflow boundary. This input boundary feature enable to exclude the entire MCR area, which reduce model run time significantly. The MCR FLO-2D model uses the breach outflow hydrograph simulated by the FLDWAV model as input as was done in the STP's RMA2 model.

At the site, the typical n-value of 0.045 was assigned. The author performed a sensitivity analysis of the MCR FLO-2D model with changing n-values from 0.025 to 0.08. The result shows that the n-value is not sensitive to the DBF levels – The change of DBF levels is only 0.36 ft which is substantially smaller than the change by breach parameters. This is because FLO-2D has a flow depth variable n-value function so that the n-values are adjusted by the model internally rather than using the input n-values.

Other model parameters for MCR, levee, and structures are the same as those used in the STP's RMA2 model. The run time of the MCR FLO-2D model is quite extensive – Simulation of a 20-hour MCR breach flood took more than 10 hours. Most of the cases, the MCR FLO-2D model simulations were stopped externally by the author after 5 hours in model time because the maximum flood levels at the site occurs 2 to 3 hours.

This study, like the STP's analysis, used a sunny day levee breach which is caused by seepage through levee or foundation, or slide of levee materials. That is, MCR antecedent rain or storm surge effects on the MCR breach flood were not considered in the MCR FLO-2D model. Evaporation and recharge losses of the MCR breach water were also ignored in flood routing as they are insignificant in determining the DBF level.

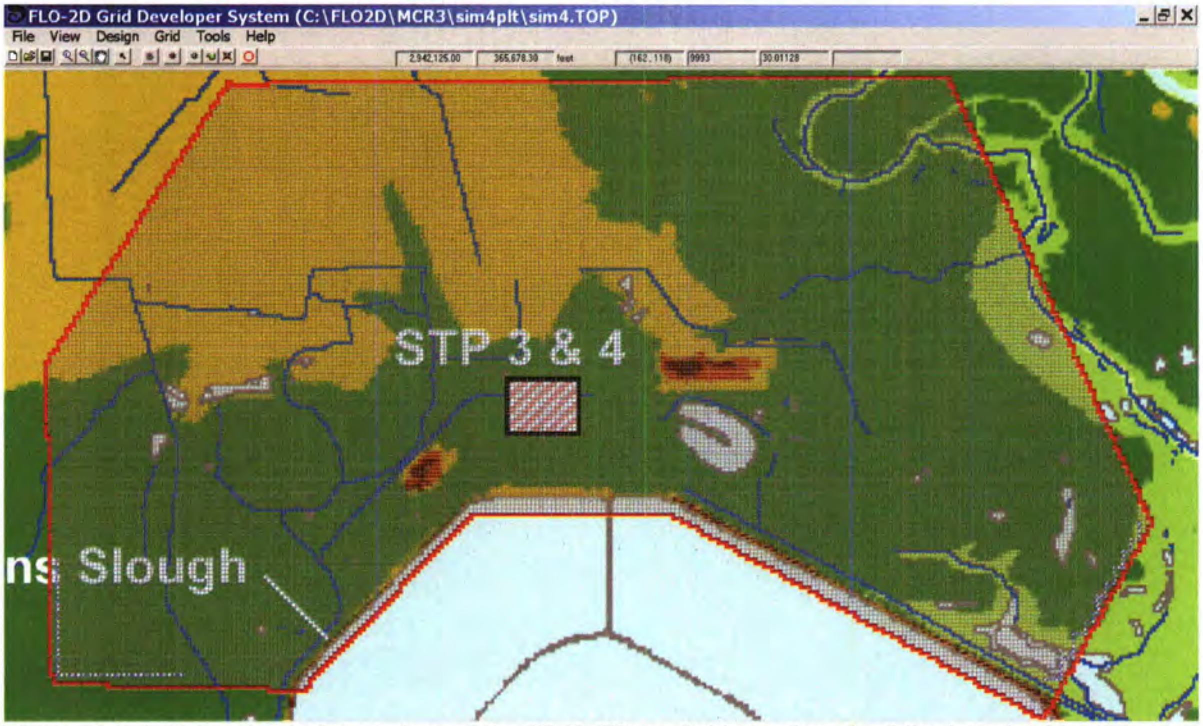


Figure 6.6-1. Two-dimensional view of the FLO-2D model boundary (red line) and grid.

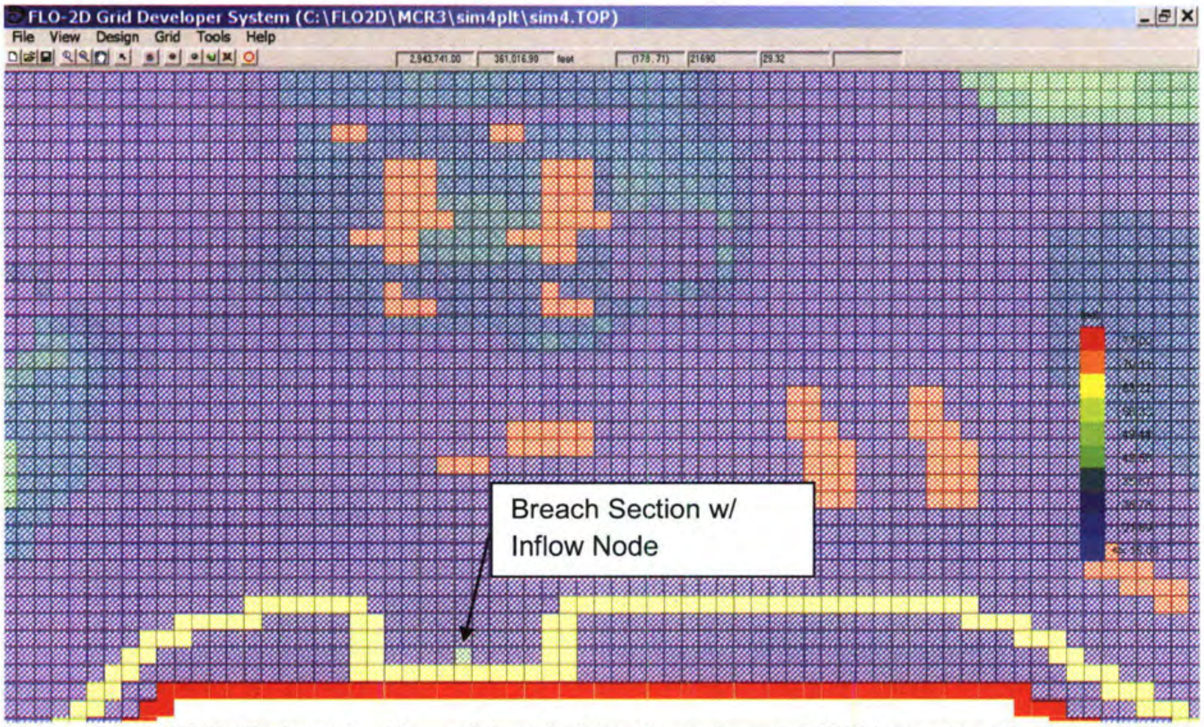


Figure 6.6-2. STP site layout with model grids (structures in red, MCR levee in yellow).

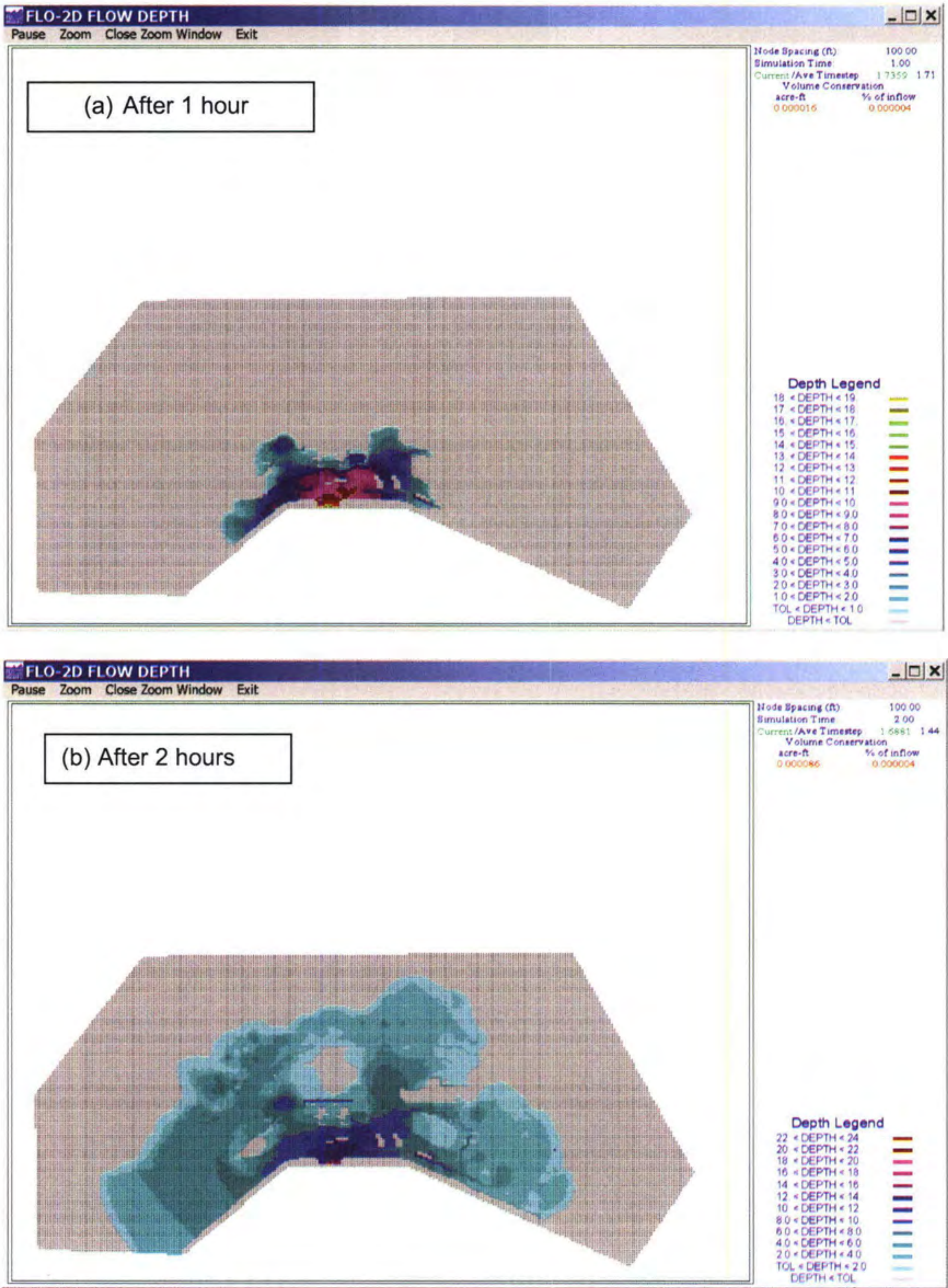


Figure 6.6-3. An example of breach flood propagations simulated by MCR FLO-2D at (a) 1 hours after initiating a breach (b) after 2 hours, and (c) after 3 hours.

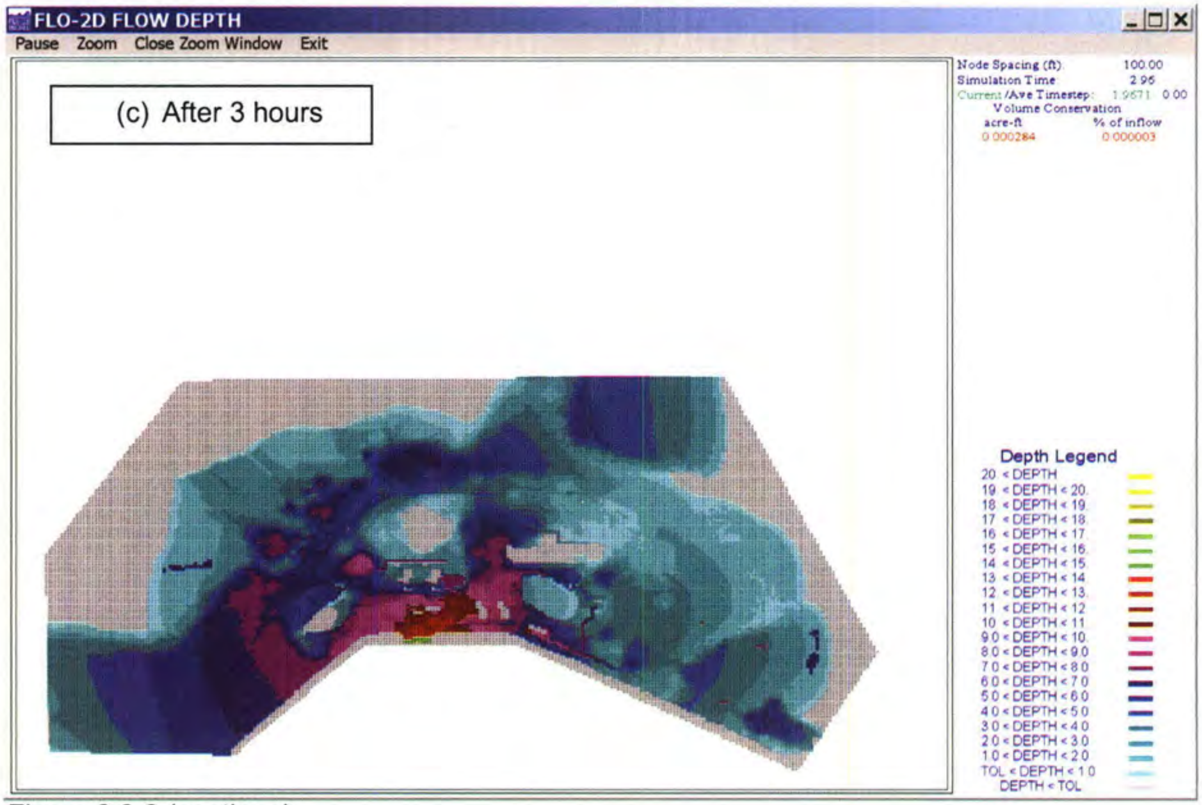


Figure 6.6-3 (continue)

This report includes some simulation maps to help understand the behavior of the MCR FLO-2D model. These maps are snapped from the FLO-2D pre/post processing software during the simulation.

Figure 6.6-3 shows a typical MCR breach flood propagation in space simulated by the MCR FLO-2D, where the propagation is displayed in terms of the maximum flow depths. This example is for the simulation of the STP's MCR breach scenario case where breach width is 417 ft and breach time is 1.7 hours. As shown from this figure, levee breach water flows towards the north with inundating the site, divides into the East and West directions almost equally, and then eventually flows to south onto either the Colorado River or the Little Robin Slough.

Figure 6.6-4 displays the land surface elevation map used in the MCR FLO-2D model. The land surface elevation data were provided by STP, which are digitized to the MCR FLO-2D model grid cells using the fLO-2D pre-processing software. Some of the cell elevations were adjusted based on the configuration of the drainage system, MCR reservoir-levee system, and proposed plant facilities to incorporate control of flow directions.

Figure 6.6-5 and 6.6-6 display simulated maximum flood elevation and maximum flood velocity maps, respectively, using the MCR FLO-2D model. The simulation condition of these maps is the same as that of Figure 6.6-3 (STP's breach scenario).

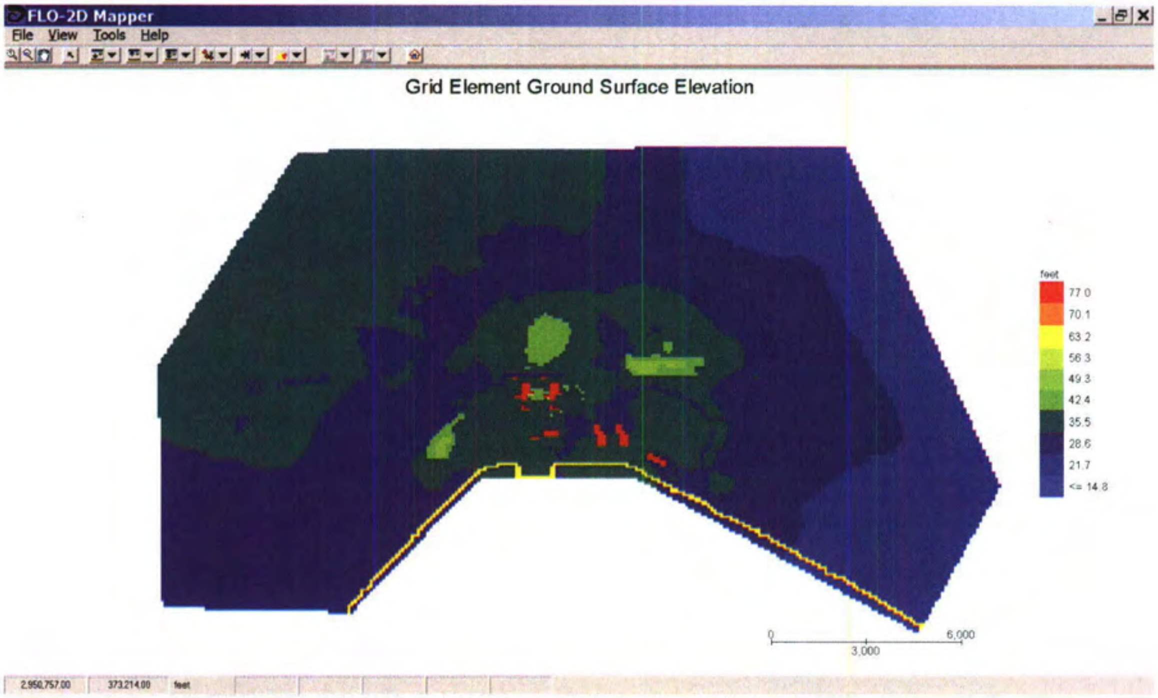


Figure 6.6-4. MCR FLO-2D surface elevation map.

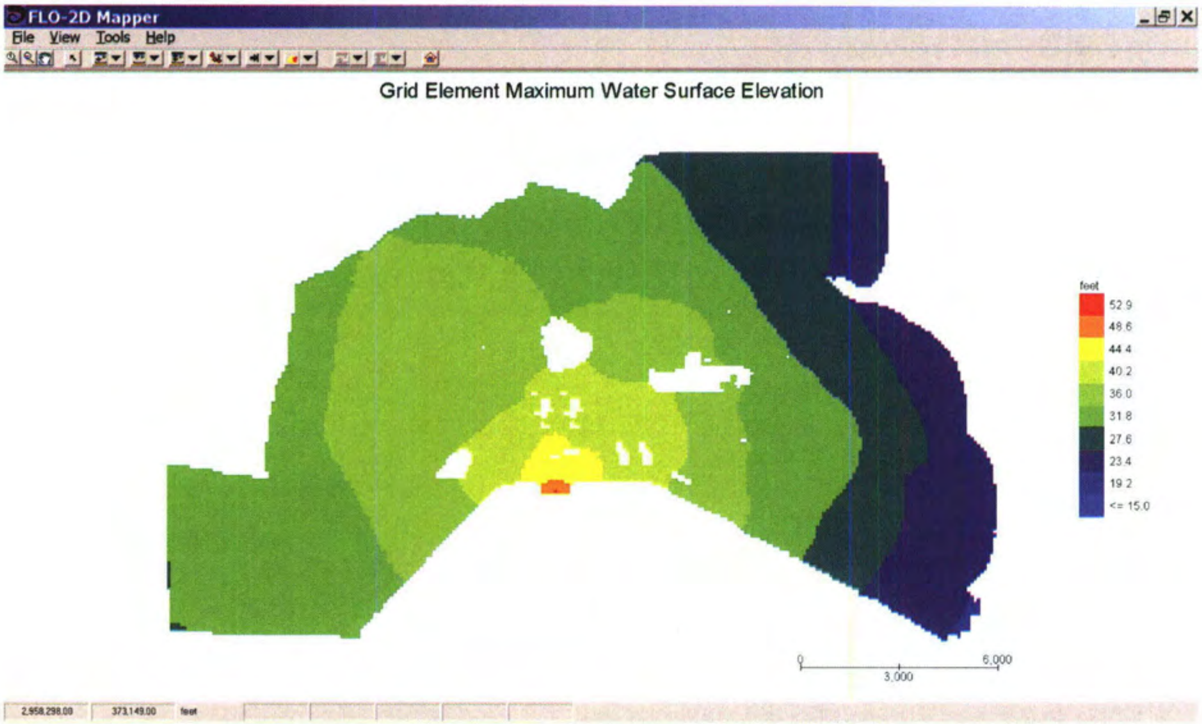


Figure 6.6-5.An example of the FLO-2D simulated maximum water surface elevation map.

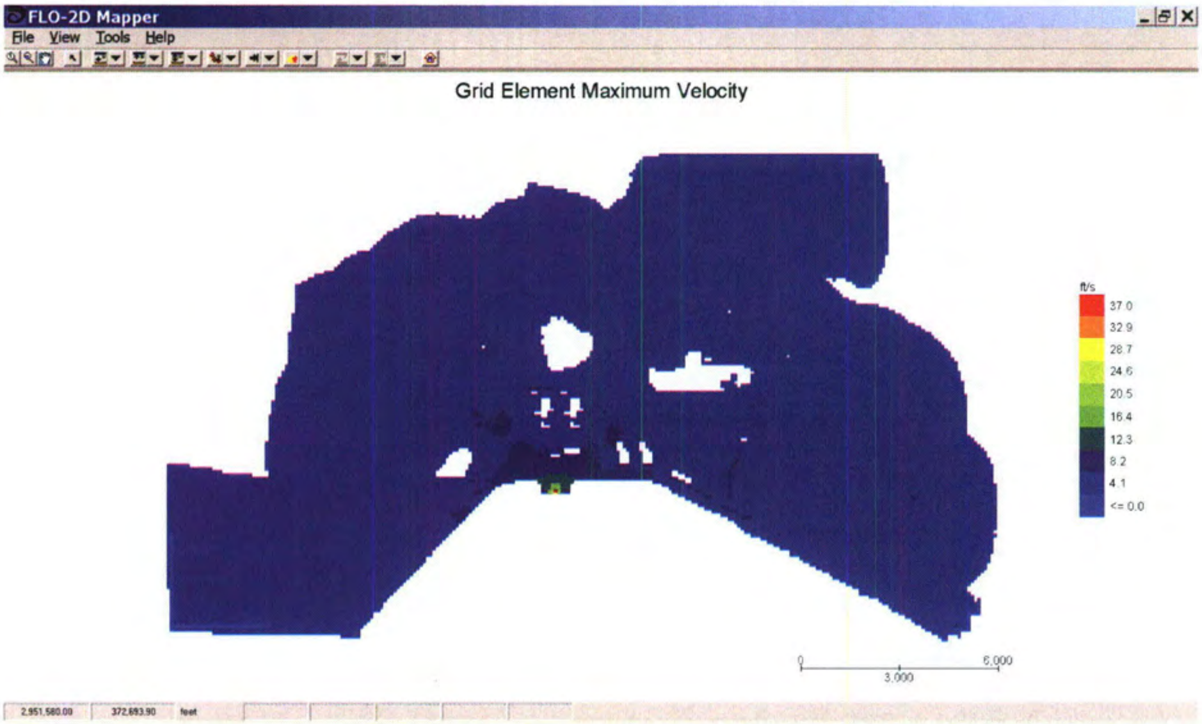


Figure 6.6-6. An example of the FLO-2D simulated maximum flood velocity map.

6.6.2 FLO-2D Model Simulation

As in the case of FLDWAV sensitivity runs, the author performed a simulation of MCR FLO-2D model for different breach widths and breach peak times. The purpose of this simulation is to set up the relation of breach scenario (characterized by breach width and peak time) and the resulting DBF level. The ranges of breach widths and peak times were determined based on the breach parameters estimated using empirical breach equations. The breach parameters were discretized (3 breach widths and 6 breach times), making a total of 18 runs. In order to monitor the simulated peak flood levels around the Units 3&4 power block area, the following 6 model nodes (node numbers) were chosen as:

17297: South of the Unit 4 Reactor Building
17553: South of the Unit 4 Service Building
18319: South of the Unit 4 UHS
17306: South of the Unit 3 Reactor Building
17562: South of the Unit 3 Service Building
18328: South of the Unit 3 UHS

On each run, the peak flood level during the simulation period (usually 5 hours) at each selected node was read from the FLO-2D output file. These peak flood level values are tabulated on the table below, and the maximum flood level (so called the DBF level) was determined from the peak flood levels at 6 nodal points. In all runs, cell number 18328 (south of the Units 3 UHS) gives the highest flood level.

Table 6.6-1. The result of simulated peak flood levels simulated by the MCR FLO-2D with different breach peak outflows and times.

Run ID	Breach width	Tp-Input	Qp	Tp	Peak Flood Level for different Cell ID						Max flood level
					17297	17553	18319	17306	17562	18328	
	ft	hr	Kcfs	hr	ft	ft	ft	ft	ft	ft	ft
b500t05	500	0.5	285	0.5	42.78	43.27	44.11	43.74	43.57	44.17	44.17
b500t10	500	1.0	277	1	42.63	43.14	43.92	43.60	43.47	44.03	44.03
b500t15	500	1.5	265	1.5	42.50	42.98	43.72	43.43	43.29	43.84	43.84
b500t20	500	2.0	241	2	42.03	42.53	43.26	42.97	42.83	43.34	43.34
b500t23	500	2.5	210	2.375	41.48	41.96	42.62	42.39	42.23	42.74	42.74
b500t27	500	3.0	189	2.7	41.01	41.47	42.11	41.89	41.74	42.19	42.19
b500t05	750	0.5	415	0.5	44.82	45.40	46.36	45.90	45.75	46.46	46.46
b500t10	750	1.0	391	1	44.62	45.12	46.06	45.62	45.47	46.19	46.19
b500t15	750	1.5	331	1.5	43.70	44.24	45.07	44.72	44.57	45.24	45.24
b500t20	750	2.0	280	1.8	42.85	43.37	44.17	43.83	43.68	44.29	44.29
b500t23	750	2.5	244	2.125	42.19	42.7	43.45	43.15	43.02	43.56	43.56
b500t27	750	3.0	219	2.4	41.67	42.16	42.87	42.60	42.44	42.93	42.93
b500t05	1000	0.5	539	0.5	46.52	47.30	48.28	47.70	47.56	48.36	48.36
b500t10	1000	1.0	466	1	45.64	46.25	47.31	46.87	46.68	47.47	47.47
b500t15	1000	1.5	369	1.35	44.35	44.88	45.80	45.38	45.29	45.95	45.95

b500t20	1000	2.0	309	1.6	43.36	43.89	44.70	44.37	44.21	44.85	44.85
b500t23	1000	2.5	271	1.875	42.74	43.23	44.01	43.71	43.54	44.15	44.15
b500t27	1000	3.0	241	2.1	42.09	42.59	43.32	43.02	42.89	43.42	43.42

6.6.3 Relation between DBF Level and Breach Parameters

Using the resulting DBF levels on the table above, the author performed a multiple linear regression analysis to obtain a regression equation to predict DBF level. The independent variable of the regression is DBF level at the Units 3 and 4 area and the dependent variables are peak breach outflow and peak time. The purpose of this regression analysis is to predict DBF level easily without re-simulating the MCR FLO-2D model that requires a substantial run-time. The resulting regression equation is as following:

$$H = 39.26042+0.01757*Qp/1000-0.03168*Tp, \text{ with } R^2=0.9895 \qquad \text{(Equ. 6.6-1)}$$

where

H is the DBF level at the Units 3&4 area in ft MSL
Qp is the peak flow in cfs, and
Tp is the peak time in hour.

The regression to predict DBF level is quite good as the regression statistics shown (refer to Table 6.6-2 and Figure 6.6-7). Qp is more sensitive to the DBF level than Tp for the possible ranges of these breach parameters. To estimate DBF levels, this equation needs peak breach outflow rate and peak time. These parameters could be estimated from empirical equations or by simulating BREACH, FLDWAV, HEC-RAS, or other breach models.

Table 6.6-2. Regression statics for predicting DBF level using peak outflow and time.

Regression Statistics					
Multiple R	0.994732				
R Square	0.989492				
Adjusted R Square	0.988091				
Standard Error	0.183661				
Observations	18				
ANOVA					
	df	*SS*	*MS*	*F*	*Significance F*
Regression	2	47.64701	23.8235	706.2698	1.44968E-15
Residual	15	0.505972	0.033731		
Total	17	48.15298			

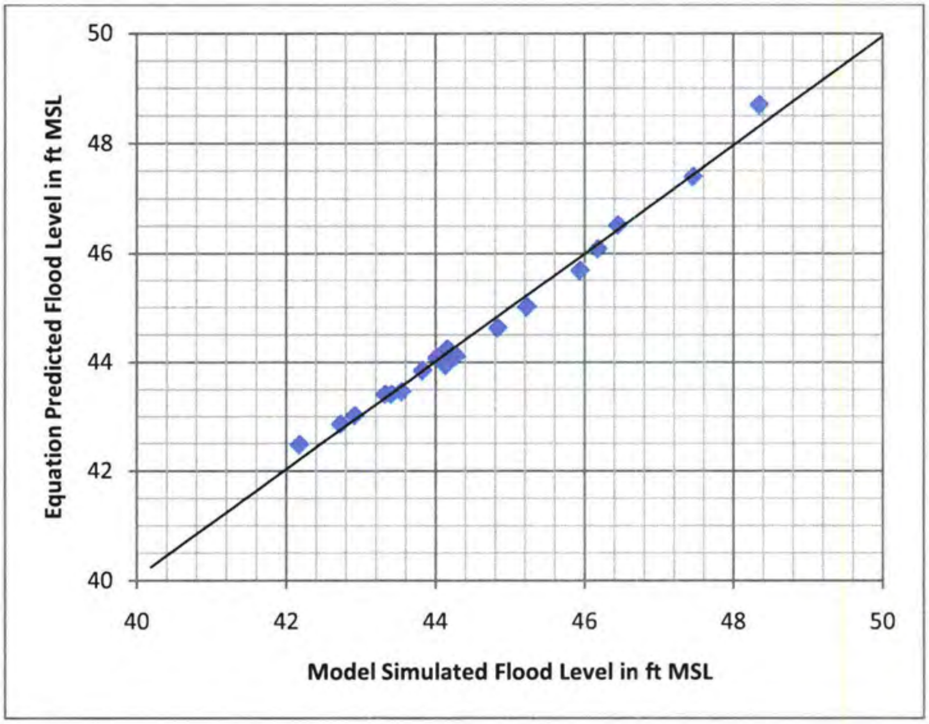


Figure 6.6-7. Linear relationship between the FLO-2D simulated DBF level and predicted one by a multiple linear regression equation.

6.6.4 Validation of FLO-2D Simulations

Direct validation of the MCR FLO-2D model is not possible because there is no actual MCR breach data. However, the model can be validated indirectly using the result of different flood routing models. For MCR breach flood routing, STP used the Delft3D model in the FSAR Revision 0 and the RMA2 model in the FSAR revision 2. The author compared the DBF level obtained by the FLO-2D simulation with those two simulations. Two MCR breach scenarios are used in the comparison: scenario for (B=417 ft, Tf=1.7 hr), and scenario for (B=4757ft, Tf=2.44 he).

STP obtained the maximum flood level of 38.9 ft MSL using the RMA2 model, then determined the DBF level of 40 ft MSL considering a margin for wind wave setup. The author simulated the FLO-2D model with the STP's breach scenario, but obtained the maximum flood level of about 40 ft MSL (without wind setup) which is about 1 foot higher than RMA2 result. The author tried to reduce the FLO-2D DBF level estimates to that of RMA2 by changing n-value, land surface elevations, structure configuration, and breach location and nodes in the model. However, the effort was fruitless nor finding any error in the MCR FLO-2D model. It is author's opinion that STP's RMA2 model could be wrong because they used the unexplained sump zone around the proposed power block area. However it is impossible for the author to check the STP's RMA2

model because the RMA2 model is not available to the staff. Instead of further efforts to refine the MCR FLO-2D model, the author attributed the one feet difference as a modeling uncertainty.

STP provided in the FSAR Revision 0 a table listing the DBF levels estimated by the Delft3D model for different MCR breach widths - They are plotted in Figure 6.6-8. Delft3D-FLOW is a multidimensional hydrodynamic and transport numerical model. This model can simulate unsteady flow and transport phenomena that result from tidal and meteorological forcing on a rectilinear or a curvilinear boundary-fitted grid system. The model solves the Navier-Stokes equations for incompressible fluid using the shallow water and the Boussinesq assumptions. The result of the Delft3D simulation for the STP breach scenario matches that by the RMA2 simulation even though the breach time for the Delft3D simulation is unknown. For the breach width of 4757 ft, FLO-2D and Delft3D gives the DBF levels of 47 ft MSL and 47.7 ft MSL, respectively.

Based on the above comparison results, it was concluded that within the model uncertainty Of 1 foot the MCR FLO-2D provides reasonable DBF level estimates for the range of possible breach parameters.

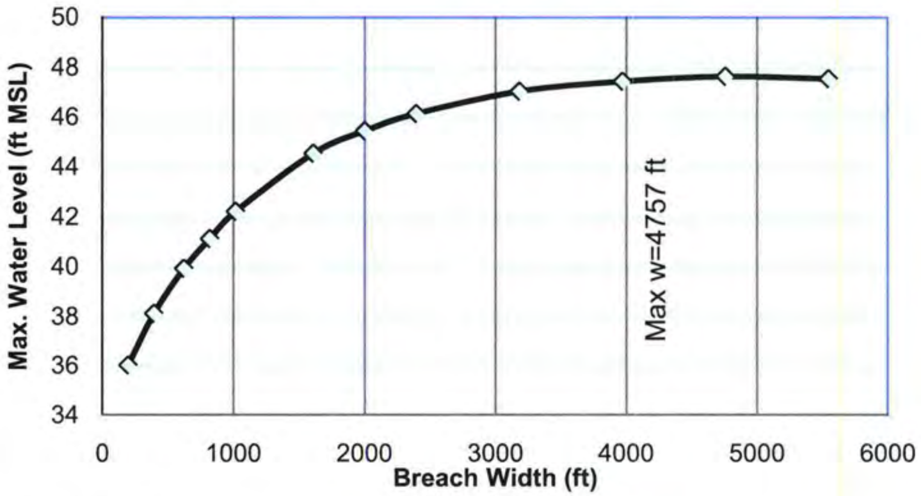


Figure 6.6-8. The relation between MCR breach width and the DBF level estimated by the STP using the Delft3D model (Based on the STP FSAR Revion 0 data).

6.6.5 Sensitivity Analysis of FLO-2D Simulation

The author performed a comprehensive sensitivity analysis for the MCR FLO-2D model. The objectives of this sensitivity analysis were to determine the effects of input variable on the DBF levels and to assure the confidence for the inputs and parameters used in FLO-2D modeling. The following input parameters were used in the sensitivity analysis:

- Model outflow boundary

- Grid size
- Manning's n-value
- Land surface elevations, especially near the site.
- Elevations for drainage ditches, Little Robin Slough, and FM521 Highway
- Location and number of the model inflow (breach outflow) nodes
- Existing and proposed structures, and
- Others

This report does not present a detailed result of these sensitivity runs because most of them are not sensitive to the DBF level compared to those for the breach width and time - The input and output files are available in the electronic format. The following is the major findings from this sensitivity test:

- Most of the cases, the model is not sensitive to the above parameters. For instance, the sensitivity analysis of Manning's n-values range from 0.02 to 0.08 (0.04 is used in the model) shows that the change of the corresponding DBF levels is only 0.34 feet which is substantially smaller than that of breach parameters. This is because the FLO-2D model uses corrected n-values based on the flow conditions and flow depths in each cells.

- The sensitivity runs for drainage ditch elevation show that they are not sensitive on the DBF levels even though they affecting the flood level at the beginning of the flood. This is because the capacities of ditches or local streams are too small to control the DBF floods.

- Also, the sensitivity results reveals that the DBL levels simulated by the FLO-2D is not sensitive to the change of the elevations near the plant area, exact locations of facilities, or breach outflow nodal conditions.

6.6.6 Estimation of DBF Levels for Different Scenarios

The author postulated 8 conservative MCR breach scenarios based on the sensitivity analyses using the BREACH, FLDWAV, and FLO-2D models. The descriptions of each scenario are followed:

- MLM-B-D10: Use the MLM breach width equation with **10 ft** scour hole depth; use the MLM breach time equation; estimate Qp and Tp from 6.5-1 and 6.5-2, respectively; n-value used.

- MLM-B-D20: Use the MLM breach width equation with **20 ft** scour hole; use the MLM breach time equation; estimate Qp and Tp from 6.5-1 and 6.5-2, respectively; n-value used.

- MLM Qp: Use the MLM Qp equation; use the MLM Tf for Tp, no scour hole, no n-value.

- Avg Qp: Same as above but use an average Qp of 251 kcfs.

- RUN1: corrected SER scenario with tailwater cross section and max levee length of 3000 ft}. Note: MCR BREACH gives a runtime error if width is greater than 3000 ft.

- RUN2: RUN 1 with a 10-ft scour hole.

- RUN23: RUN 1 with a 15ft scour hole.

- RUN24: RUN 1 with a 20-ft scour hole.

Once Qp and Tp were estimated from either empirical equations or breach models, DBF levels are computed using Equation 6.6-1. The table below lists the breach parameters as well as estimated DBF levels for each scenario. The result of this estimation indicates that the reasonably conservative DBF levels at the proposed Units 3 & 4 is about 45 ft MSL considering a margin for wind wave setup.

The DBF level estimates in this table are based on the following three different approaches:
- MPM breach width equation, FLDWAV, and FIO-2D
- MLM peak flow, FLO-2D
- BREACH, FLO-2D

For the BREACH model, n-value of 0.075 is used. The DBF level estimates for the first four scenarios were not relied on the Manning's n-value, but use different empirical equations and FLDWAV. All these approaches produced very consistent DBF estimates.

Table 6.6-3. DBF levels estimated by Equation 6.6-1 with various MCR breach scenarios.

Run ID	B (ft)	Tf (hr)	Qp (kcfs)	Tp (hr)	DBF level H (ft MSL)	Remark
MLM-B-D10	1047	2.54	309	1.87	44.63	Qp &Tp from FLDWAV,10 ft hole
MLM-B-D20	745	2.54	280	2.12	44.11	Qp &Tp from FLDWAV, 20 ft hole
MLM Qp	-	-	217	2.54	42.99	Qp &Tp from FLDWAV, 0 ft hole
Avg Qp	-	-	251	2.54	43.59	Qp &Tp from FLDWAV, 0 ft hole
RUN1	934	-	194	3.29	42.56	Qp &Tp from BREACH, 0 ft hole
RUN2	633	-	269	2.12	43.92	Qp &Tp from BREACH, 10 ft hole
RUN23	516	-	271	1.8	43.96	Qp &Tp from BREACH, 15 ft hole
RUN24	433	-	267	1.64	43.90	Qp &Tp from BREACH, 20 ft hole

6.6.7 Very Extreme MCR Breach Scenarios

The breach parameter estimates in Subsection 6.3 are based on the best-fitted linear regression line shown (dot lines in Figure 6.6-9). The problem in using the best-fitted line is that about a half of historical breach records would exceed the best-fit estimate – This expected value result in an underestimation. If an extreme conservatism is of concern, one may use a bounding (dashed envelop) line equations. On this regard, the following three extreme scenarios were postulated:

- MLM-B-D0: Use MLM breach width equation with no scour hole, then use FLDWAV to estimate Qp and Tp. It will end up a wider breach width than one with scour hole.
- Extreme B: Use a STP postulated breach width of 4745 ft in the FSAR Revision 0, then use FLDWAV to estimate Qp and Tp. This may be the case where a bounding MLM breach width equation is used.
- High Qp: Use an average of 5 high Qp's estimated using empirical equations (Qp=336 kcfs).

As before, DBF levels were computed by Equation 6.6-1 with estimated Qp and Tp. The table below lists the breach parameters as well as estimated DBF levels for each scenario. The result of this estimation shows that the extremely conservative DBF level would be about 47 ft MSL, where as FSAR Revision 0 DBF level estimate based on the simulation of the Delft3D model was is 47.6 ft MSL.

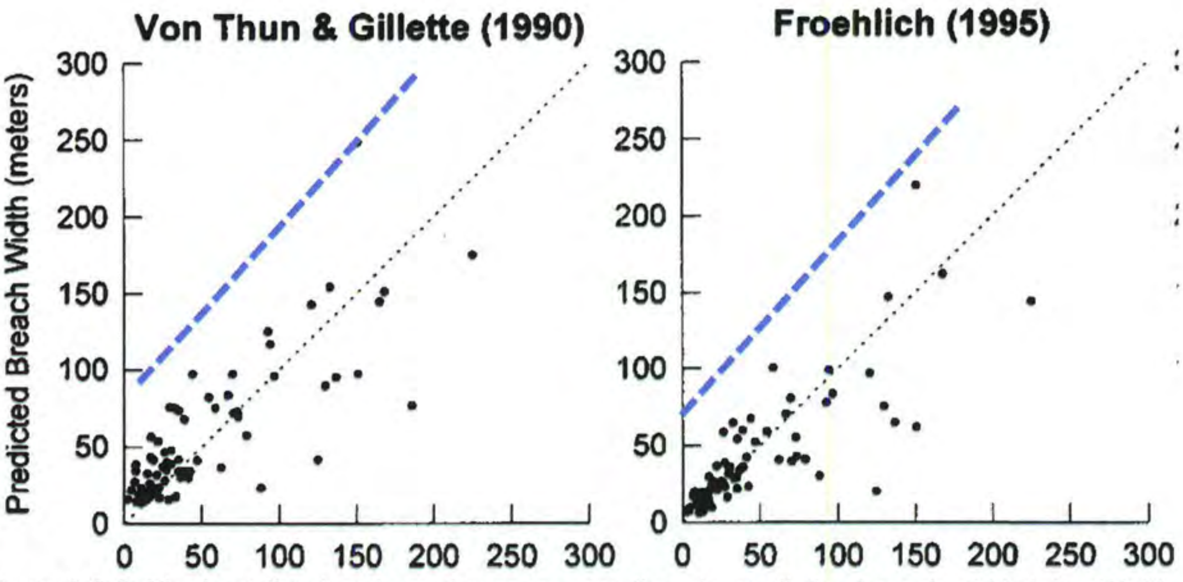


Figure 6.6-9. The best-fitted and envelop regression lines in predicting breach width (after Wahl, 1998).

Table 6.6-4. DBF levels estimated by Equation 6.6-1 with extremely conservative MCR breach scenarios.

Run ID	B (ft)	Tf (hr)	Qp (kcfs)	Tp (hr)	DBF level H (ft MSL)	Remark
MLM-B-D0	1736	2.54	380	1.55	45.89	0 ft hole
Extreme B	4745	2.54	430	1.3	46.77	0 ft hole
High Qp	-	1.7	336	1.3	45.12	MLM envelop Tf

6.7 Frequency of Occurrence of MCR Breach

FSAR 2.4 dam breach analysis is based on a deterministic approach that does not consider the condition of the levee or frequency of failure occurrence. However, this section includes an investigation of the probability of the MCR breach failure as referencing information. Generic dam failure frequency which is mainly based on small or medium size dams (length of less than 2000 ft) cannot be applicable to MRC breach. The above generic failure probability cannot be used because a total MCR levee length is 12.6 miles. The MCR breach failure frequency could be higher than the generic one. This report uses the following three sources of information:

- Foster et al. (2000: Table 5) said that an average annual frequency of embankment levee failure (earth-fill levee with filter) due to piping is about $1.89\text{x}10^{-4}$/year for first 5 years of operation and $0.37\text{x}10^{-4}$ /year after 5 years of operation.
- The frequency of levee failures for the California Delta Levee System (a total levee length of 911 miles) is about $1.18\text{x}10^{-4}$/year/levee mile.
- The frequency of the Suisun Marsh levee (a total of 75 miles) is about $4.76\text{x}10^{4}$/year/levee mile.

In summary, the estimated frequency of MCR levee failure is an order of **10^{-4}/year.** However, this estimate needs to be corrected considering the length and condition of the MCR levee.

7 Conclusion

The author of this report performed a re-analysis of the main cooling reservoir (MCR) breach flood as part of reviewing the South Texas Project combined license application. The result of the re-analysis proves that both STP's and PNNL's MCR breach flood analyses are inaccurate and non-conservative> they underestimated the MCR breach parameters (breach width and peak outflow rate) and the resulting design basis flood level. The author's re-analysis reveals that the reasonably conservative design basis flood level would be about 45 ft mean sea level, or it could exceed about 47 ft mean sea level if a very conservative MCR breach scenario is considered. Because the design basis flood level is critical in safety-related structural designs and flood protection, the author recommends revising the dam breach sections of FSAR and the current version of the SER using a correct and conservative MCR breach scenario.

References

- Acrement G. J. and V. R. Schneider, 1984, Guide for Selecting Manning's Roughness Coefficients for Natural Channel and Flood Plains, USGS Water Supply Paper 2339.

- Baecher, G.B., M. Elisabeth, and R. D., Neufville, 1980, "Risk of Dam Failure in Benefit-Cost Analysis, Water Resources Research, Vol. 16, No. 3, Pages 449-456.

- Brunner, G. W., 2007, Using HEC-RAS to Perform a Dam Break Analysis, a paper distributed at the FY07 Training for Hydrology and Hydraulics for Dam Dafety Studies by USACE/ HEC, Davis, CA.

- Chauhan, S.S., D.S. Bowles, and L.R. Anderson, 2004, ASDSO Proceeding of Dam Safety, "Do Current Breach Parameter Estimation Techniques Provide Reasonable Estimates for Use in Breach Modeling?" 2004 ASDSO Proceeding of Dam Safety, Utah State University and RAC Engineers & Economics, Logan, UT.

- Chow, V.T. 1959. Open Channel Hydraulics. McGraw-Hill, New York.

- FLD-2D Software, Inc., 2006, FLO-2D User Manual, Version 2006.01.

- Foster, M., R. Fell, and M. Spannagle, 2000, "The Statistics of Embankment Dam Failures and Accidents", Can. Geotech. J. 37:1000-1024.

- Fread, D.L., 1988, The NWS-DMABRK Model, Quick User Guide, Revision 4, 1991, Hydrologic Research Laboratory, Office of Hydrology, National Weather Service, NOAA

- Fread, D.L. 1991. BREACH: An Erosion Model for Earthen Dam Failures. Hydrologic Research

- Fread, D.L., 1988 (revised in 1991), BREACH, An Erosion Model for Earthen Dam Failures, National Weather Service, National Oceanic and Atmospheric Administration, Silver Spring, MD.

- Fread, D.L., 1993, NWS FLDWAV Model: The Replacement of DAMBRK for Dam-Break Flood Prediction, Dam Safety '93, Proceeding of the 10th Annual ASDSO Conference, Kansas City, Missouri.

- Fread, D.L., 2000. NWS FLDWAV Model: Theoretical Description," Hydrologic Research Laboratory, Office of Hydrology, National Weather Service, NOAA (Revision 4 1991).

- Froehlich, D. L. 1995a. Embankment dam breach parameters revisited. Water Resources Engineering, Proceedings of the 1995 ASCE Conference on Water Resources Engineering, New York, 887–891.

- Froehlich, D. C. 1995b. Peak outflow from breached embankment dam. Journal of Water Resources Planning and Management, American Society of Civil Engineers, 121(1), 90–97.

- Gee, D.M.,.2008, Comparison of Dam Breach Parameter Estimators, HEC Report, U.S. Corps of Engineers Hydrologic Engineering Center, Davis, CA.

- Gee, D.M., 2010, Use of Breach Process Models to Estimate HEC-RAS Dam Break Parameters, 2nd Joint Federal Interagency Conference, Las Vegas, NV.

- Hydrologic Engineering Center (HEC), 2008, HEC-RASRiver Analysis Sytem, User's Manual, Version 4.0, USACE, Davis, CA.

- MacDonald, T. C., and Langridge-Monopolis, J. 1984. Breaching characteristics of dam failures. Journal of Hydraulic Engineering, 110(5), 567–586.

- Nagy, L., 2006, Estimating Dyke Breach Length from Historical Data, Periodics Polytechnica Ser. Civ. Eng. Vol 50, No. 2, pp125-139.

- Pacific Northwest National Laboratory, 2011, Calculation Worksheet: Confirm MCR embankment breach flood discharge and its sensitivity to breach parameter, submitted to NRC as a supplemental document for SER, Richmond, WA.

- SFWMD (South Florida Water Management District), 1980, Interim Final Draft Report on Embankment Failure Florida Power $ Light Company Martin Plant Cooling Reservoir, SFWMD, West Palm Beach, FL.

- STP (South Texas Nuclear Operating Company), 2007, South Texas Combined License Application, Revision 0, Part 2, Final Safety Analysis Report, Bay City, TX.

- STP (South Texas Nuclear Operating Company), 2009, South Texas Combined License Application, Revision 2, Part 2, Final Safety Analysis Report, Bay City, TX.

- STPEGS, 2006, Updated Final Safety Analysis Report (UFSAR for Units 1 & 2, Revision 13.

- U.S. Army Corps of Engineers, 2005, User's Guide to RMA2 WES, Version 4.5, Coastal and Hydraulics Laboratory, Waterway Experiment Station, Engineering Research and Development Center, Vicksburg, MS.

- U.S. Army Corps of Engineers, 2006, "Engineering and Design: Reliability Analysis and Risk Assessment for Seepage and Slope Stability Failure Modes for Embankment Dams", Department of Army, U.S. Army Corps of Engineers, ETL 1110-2-561, Washington, D.C.

- United States Geological Survey 2009. Water Resources of Illinois: n-Values Project. URL: http://il.water.usgs.gov/proj/nvalues/equations.shtml?equation=05-strickler.

- U.S. Bureau of Reclamation 1982. Guidelines for defining inundated areas downstream from Bureau of Reclamation dams, Reclamation Planning Instruction No. 82-11, U.S. Department of the Interior, Bureau of Reclamation, Denver, Colorado, 25p.
- U.S. Bureau of Reclamation 1988. Downstream hazard classification guidelines. ACER Technical Memorandum No. 11, U.S. Department of the Interior, Bureau of Reclamation, Denver, Colorado, 57p.
- U.S. Bureau of Reclamation, 1998, Prediction of Embankment Dam Breach Parameters: A Literature Review and Needs Assessment", DSO-98-004, Dam Safety Research Report, U.S. Department of the Interior, Bureau of Reclamation, Dam Safety Office.
- URS, 2007, "Technical Memorandum Delta Risk management Strategy (DRMS) Phase 1: Topical Area Levee Vulnerability Draft 2, URS Corporation/J.R. Benjamin & Associates, Inc., Oakland, CA.
- Von Thum, J.L. and D.R. Gillette, 1990, Guidance on Breach Parameters, unpublished internal document, U.S. Bureau of Reclamation, Denver, CO.
- Wahl, T.L. 2004. Uncertainty of Predictions of Embankment Dam Breach Parameters. Journal of Hydrology, 130(5), 389-397.
- Wahl, T.L., 1997, Predicting Embankment Dam Breach Parameters – A Needs Assessment, XXVIIth IAHR Congress, San Francisco, CA.
- Wahl, Tony L. , Gregory J. Hanson, Jean-Robert Courivaud, Mark W. Morris, René Kahawita, Jeffrey T. McClenathan, and D. Michael Gee, 2008. Development of Next-Generation Embankment Dam Breach Models. U.S. Society on Dams, 2008 Annual Meeting and Conference, April 28-May 2, Portland, Oregon, [online paper].
- Zhang X. and Wang, G, 2001, Flow Analysis and Scoring Hole Computation of Dyke-Breach," Proceeding of the International Association for hydraulic Researchers XXIX Congress, Theme E. Tsinghua University, Beijing, China.

Attachment 11c: Enclosure 2: PNNL's Calculation Package/Worksheet, commented on by H. Ahn June 20, 2011

Calculation Worksheet

Title: Confirm MCR embankment breach flood discharge and its sensitivity to breach parameters			**Project**: PNNL Project #
Prepared By: LF Hibler, R Prasad, and Y Yuan	**Date**: 3/16/2011	**Reviewed By**: SA Breithaupt	**Date**:

- **Purpose**:
  - Perform estimation of main cooling reservoir (MCR) embankment breach parameters using available and applicable empirical equations, the National Weather Service (NWS) BREACH model, and evaluate FLDWAV runs to confirm the applicant's assessment of the MCR embankment failure flood at the STP site. Perform sensitivity studies on the BREACH model.

- **Methods Used**:
  - The applicant used a combination of the MacDonald and Langridge-Monopolis (1984) (MLM) and Froehlich (1995a, b) approaches to develop estimates of the MCR breach parameters.
  - We evaluated the applicant's estimates of MCR embankment breach parameters using several approaches. These approaches included:
    - MLM (1984) empirical approach
    - Froehlich (1995a, b) empirical approach
    - NWS BREACH model
    - FLDWAV model

- **Assumptions & Constraints**:
  - **Assumption 1**: The predicted volume of eroded material (Ve*) in the MLM approach is the volume of embankment material, not the volume of solid particles.
  - **Verification of Assumption 1**: To verify Assumption 1, we used the Apishapa Dam data reported by MacDonald and Langridge-Monopolis (1984). The volume of the breach (V_t) was computed based on the breach height (H), top width (T_w), breach width at top (B_t), upstream and downstream side slopes of the embankment (Se1 and Se2). The computation was repeated except that breach width at bottom (B_b) rather than the breach width at top and side slopes of the breach (S_{b1} and S_{b2}) were used to estimated the volume (Vb). The two estimated volumes, Vt and Vb, were averaged ($V_a=(V_t+V_b)/2$) and the average volume was compared with the volume (V_e^*) reported in by MacDonald and Langridge-Monopolis (1984). A relative difference was computed ($RD=100^*(V_e^*-V_a)/V_e^*$).

    MacDonald and Langridge-Monopolis (1984) reported the following values:

    V_e^* = 291,000 cubic yards
    H = 100 ft
    B_t = 320 ft
    T_w = 16 ft

$S_{e1} = 1//3$
$S_{e2} = 1/2$
$S_{b1} = 6.7$
$S_{b2} = 2.9$.

We calculated

$V_t = B_t [T_w H + 0.5 H^2 (1/S_{e1} + 1/S_{e2})]/27 = 315,259$ cubic yards
$V_b = (B_t - H/S_{b1} - H/S_{b2}) [T_w H + 0.5 H^2 (1/S_{e1} + 1/S_{e2})]/27 = 266,583$ cubic yards
$V_a = 0.5 (V_t + V_b) = 290,921$ cubic yards
$RD = 100 (V_e^* - V_a)/V_e^* = 0.027$ percent.

Because the relative difference is small, Assumption 1 is confirmed. The MLM-predicted volumes should be interpreted as embankment or dam volumes rather than solid material volumes.

- **Calculation Basis**:
  - We used data provided by South Texas Nuclear Operating Company (STPNOC) in Units 3 and 4 Final Safety Analysis Report (FSAR), a part of the Combined License (COL) application to the U.S. Nuclear Regulatory Commission, responses to various Request for Additional Information (RAI), and technical reports prepared by STPNOC's contractors.

    The methods used in this calculation worksheet are described in the references listed below. We describe our assumptions, verification of these assumptions where appropriate, and steps used in these calculations.

    We ran the NWS BREACH model using the input file provided by STPNOC. We were able to reproduce the results of the NWS BREACH model reported by STPNOC in response to RAI 02.04.04-14 (STPNOC 2010). Several variations of BREACH parameters were used to investigate the sensitivity of model predictions.

- **Calculation Input**:
  - **Estimation of MCR embankment breach parameters**:
    - NWS BREACH model
    - Empirical approaches (see Wahl 2004)
  - **Estimation of discharge hydrograph following the MCR embankment breach**:
    - Based on the BREACH sensitivity analysis, we selected a conservative set of parameters that is expected to result in conservative predictions of breach size and peak discharge.
  - **Sensitivity analysis for embankment breach parameters**:
    - NWS BREACH model
      - Sensitivity of the model to elevation at which piping failure commences, Z_p (ft)
      - To evaluate the sensitivity of the model to Z_p, we used the following values: $Z_p = \{ 29, 30, 31, 32, 33, 34, 35, 36, 37, 38, 39, 40, 41, 42, 43, 44, 45 \}$ ft
      - The applicant used a Z_p of 34 ft MSL (STPNOC 2010).
      - Sensitivity of the model to Manning's n of dam material

- To evaluate the sensitivity of the model to n, we used the following values: n = { 0.007, 0.008, 0.010, 0.015, 0.020, 0.025, 0.030, 0.040, 0.050, 0.060, 0.070, 0.080 }
- The applicant used a Manning's n value of 0.05 (STPNOC 2010). Strickler's equation for prediction of Manning's n from median grain size is (Fread, 1991)

  $$n = 0.013\ (D_{50})^{0.67}$$

  with D_{50} in mm. Using the applicant's D_{50} value of 0.001 mm, n is estimated as 0.0001. The applicant stated that this estimate was too low and used a value of 0.05 for Manning's n (STPNOC 2010).

  Strickler's equation is also given by USGS (2009). The USGS (2009) form of the equation is

  $$n = 0.015\ (D_{50})^{(1/6)}$$

  USACE (1994) also indicates that the exponent in Strickler's equation should be 1/6 or 0.167, but the units for D_{50} is ft:

  $$n = 0.034\ (D_{50})^{(1/6)}$$

  These n-values are too small - need justification.

  Using the three forms of Strickler's equation, we estimated the Manning's n as 0.0001 (Fread 1991 form), 0.005 (USGS 2009 form), and 0.004 (USACE 1994 form).

- Sensitivity of the model to cohesive strength of dam material, C (lb/ft^2)
  - To evaluate the sensitivity of the model to C, we used the following values: C = { 50, 100, 150, 200, 250, 300, 350, 400 } lb/ft^2
  - The applicant used a C of 300 lb/ft^2. The applicant's chosen value is the minimum of the reported undrained and drained cohesive strengths of the embankment material (Bechtel Energy Corporation 1984).

- Sensitivity of the model to internal friction angle of dam material, Ø (degrees)
  - To evaluate the sensitivity of the model to internal friction angle, we used the following values: Ø = { 10, 15, 20, 25, 30 } degrees
  - The applicant used a Ø of 20 degrees. Bechtel Energy Corporation (1984) reported values of Ø to be 16 and 20 degrees for the undrained and drained embankment material.

- Sensitivity of the model to length of dam, L (ft)
  - To evaluate the sensitivity of the model to the length of the dam, we used the following values: L = { 1000, 2000, 3000, 4000 } ft
  - The applicant used an L of 1000 ft. The east-to-west running portion of the north face of the MCR embankment is approximately 4300 ft in length.

- Empirical approaches
  - Depth of water above breach invert at time of failure, h_w = 21.9 ft (6.68 m)
  - Volume of water stored above breach invert at time of failure, V_w = 152,700 ac-ft (188,352,670.9 m^3)

- Height of breach, h_b = 37 ft (11.28 m)
- Overtopping multiplier, K_o = 1.4 for overtopping, 1.0 for piping

- **References**:

  - **Bechtel Energy Corporation 1984**. Evaluation of Strength Parameters and Stability Main Cooling Reservoir Embankment. Volume 1 Report. Harza Engineering Company, Houston, Texas.
  - **Chow, V.T. 1959**. Open Channel Hydraulics. McGraw-Hill, New York.
  - **Fread, D.L. 1991**. BREACH: An erosion model for earthern dam failures. Hydrologic Research Laboratory, Office of Hydrology, National Weather Service, U.S. National Oceanic and Atmospheric Agency, Silver Spring, Maryland, July, 1991.
  - **Froehlich, D. C. 1995a**. Embankment dam breach parameters revisited. Water Resources Engineering, Proceedings of the 1995 ASCE Conference on Water Resources Engineering, New York, 887–891.
  - **Froehlich, D. C. 1995b**. Peak outflow from breached embankment dam. Journal of Water Resources Planning and Management, American Society of Civil Engineers, 121(1), 90–97.
  - **MacDonald, T. C., and Langridge-Monopolis, J. 1984**. Breaching characteristics of dam failures. Journal of Hydraulic Engineering, 110(5), 567–586.
  - **Merz, B. and A.H. Thieken 2005**. Separating natural and epistemic uncertainty in flood frequency analysis. Joural of Hydrology, 309(1-4), 114-132.
  - **STPNOC 2010**. Letter from Scott Head to NRC Document Control Desk, U7-C-STP-NRC-100241, November 22, 2010.
  - **U.S. Army Corps of Engineers 1994**. Hydraulic Design of Flood Control Channels. Engineer Manual EM 1110-2-1601, Washington DC.
  - **U.S. Bureau of Reclamation 1982**. Guidelines for defining inundated areas downstream from Bureau of Reclamation dams, Reclamation Planning Instruction No. 82-11, U.S. Department of the Interior, Bureau of Reclamation, Denver, Colorado, 25p.
  - **U.S. Bureau of Reclamation 1988**. Downstream hazard classification guidelines. ACER Technical Memorandum No. 11, U.S. Department of the Interior, Bureau of Reclamation, Denver, Colorado, 57p.
  - **United States Geological Survey 2009**. Water Resources of Illinois: n-Values Project. URL: http://il.water.usgs.gov/proj/nvalues/equations.shtml?equation=05-strickler.
  - **Wahl, T.L. 2004**. Uncertainty of Predictions of Embankment Dam Breach Parameters. Journal of Hydrology, 130(5), 389-397.

- **Units**:
  - Some of the estimation equations and model input data used SI units. When original data were available in English units, we used equivalent values converted to SI units for these inputs. The final results from the calculations are reported in English units after conversion.

- **Results**:
  - **Sensitivity analysis for embankment breach parameters**:
    - NWS BREACH model
      - Sensitivity of the model predictions to elevation at which piping failure commences, Z_p (ft). The model predictions are fairly insensitive to Z_p. At Z_p of 30 ft and lower, the model run did not complete because of a mathematical error, probably a result of Z_p being too close to the bottom of the reservoir. At Z_p of 46 ft and higher, which is very close to the top of the initial water surface elevation in

the reservoir, it appears the breach did not develop fully to erode a large portion of the embankment.

Z_p (ft MSL)	Q_p (cfs)	B_t at Peak Flow (ft)	Bb at Peak Flow (ft)	T_p (hr)
29	--	--	--	--
30	--	--	--	--
31	83,261	412.5	360.7	6.1
32	83,239	412.4	360.6	6.2
33	83,216	412.9	361.1	6.2
34	83,197	412.4	360.6	6.3
35	83,176	412.1	360.3	6.3
36	83,152	412.3	360.5	6.4
37	83,175	412.1	360.3	6.4
38	83,200	412.4	360.6	6.4
39	83,074	411.1	359.3	6.7
40	83,089	411.3	359.5	6.7
41	83,101	411.3	359.5	6.8
42	83,110	412.3	360.5	6.9
43	83,123	411.8	359.9	7.1
44	83,137	411.7	359.9	7.4
45	83,150	413.1	361.3	12.7
~~46~~	~~416~~	~~4.9~~	~~4.9~~	~~0.0~~
~~47~~	~~238~~	~~3.9~~	~~3.9~~	~~0.0~~

These are model errors. Should delete them

- Sensitivity of the model predictions to Manning's n of dam material. The model predictions are very sensitive to Manning's n; however, it should be noted that the range of variation of Manning's n covers three orders of magnitude. The applicant, using Strickler's equation obtained an estimate of 0.0001 for Manning's n with a D_{50} of 0.001 mm. Using the three forms of Strickler's equation presented above, we estimated the Manning's n as 0.0001 (Fread 1991 form), 0.005

(USGS 2009 form), and 0.004 (USACE 1994 form). For comparison, recommended lowest Manning's n for smooth brass, Lucite, and glass channels flowing partially full are at least nearly two times greater at 0.009, 0.008, and 0.009, respectively (Chow 1959). To estimate the sensitivy of model predictions we varied n from 0.0001 to 0.1, a range of three orders in magnitude, to accommodate the applicant's estimate of n, reasonable values of n reported in literature, and a conservatively selected upper range.

n	Q_p (cfs)	B_t at Peak Flow (ft)	B_b at Peak Flow (ft)	T_p (hr)
0.0001	445	3.7	3.7	1.25
0.001	2	0.3	0.3	1.25
0.005	876	5.2	5.2	1.25
0.008	4,199	11.4	11.4	1.25
0.010	8,424	16.4	16.4	1.25
0.015	15,833	110.4	58.6	23.7
0.020	22,727	144.5	92.7	19.6
0.025	30,762	183.6	131.8	15.9
0.030	48700 ~~39,776~~	317 ~~229.7~~	206 ~~177.9~~	10.5 ~~13.3~~
0.040	73800 ~~60,500~~	438 ~~329.1~~	327 ~~277.3~~	7.55 ~~9.4~~
0.050	103000 ~~83,197~~	580 ~~412.4~~	470 ~~360.6~~	5.79 ~~6.3~~
0.060	136000 ~~99,695~~	739 ~~449.1~~	629 ~~397.3~~	4.63 ~~4.1~~
0.070	172000 ~~112,518~~	917 ~~481.2~~	806 ~~429.4~~	3.85 ~~2.9~~
0.080	207000 ~~122,806~~	1017 ~~516.6~~	906 ~~464.8~~	2.93 ~~2.4~~
0.100	252000 ~~136,945~~	1093 ~~564.3~~	983 ~~512.5~~	1.82 ~~1.68~~

PNNL set up the BREACH model incorrectly so that breach width and outflow are constricted by small levee width and tailwater channel width.
Red ones are corrected values with levee width and tailwater section of 3000 ft

Sensitivity of the model predictions to cohesive strength, C, of dam material. The model predictions are no sensitive to values of C ranging from 250 to 400 lb/ft^2. As values of C were lowered from 250 lb/ft^2, peak discharge and breach width increased and the time to peak decreased. However, even with a very low cohesive strength of 50 lb/ft^2, the embankment breach width at peak flow was approximately 512 ft.

C (lb/ft^2)	Q_p (cfs)	B_t at Peak Flow (ft)	B_b at Peak Flow (ft)	T_p (hr)

50	99,628	512.3	377.4	4.35
100	92,009	460.9	364.4	4.93
150	92,009	460.9	364.4	4.93
200	87,013	460.4	363.9	6.01
250	83,197	412.4	360.6	6.25
300	83,197	412.4	360.6	6.25
350	83,197	412.4	360.6	6.25
400	83,197	412.4	360.6	6.25

- Sensitivity of the model predictions to internal friction angle, Ø, of dam material. The model predictions are only slightly sensitive to this parameter.

Ø (degrees)	Q_p (cfs)	B_t at Peak Flow (ft)	B_b at Peak Flow (ft)	T_p (hr)
10	84,571	415.9	353.8	5.76
15	84,143	413.8	357.0	5.95
20	83,197	412.4	360.6	6.25
25	82,012	403.0	355.9	6.31
30	80,959	394.6	351.9	6.36

- Sensitivity of the model predictions to length of dam, L (ft). Because the east-to-west running portion of the north face of the MCR embankment is approximately 4300 ft in length, we limited the dam length to 4000 ft. ~~Model prediction were not at all sensitive to L values ranging from 1000 to 4000 ft.~~

This conclusion is incorrect. Model predictions were very sensitive to L for the range from 400 ft to 1000 as shown on the next table.

L (ft)	Q_p	B_t at Peak Flow	B_b at Peak Flow	T_p
1000	207000 ~~83,197~~	1000 ~~412.4~~	889 ~~360.6~~	2.85 ~~6.25~~
2000	207000 ~~83,197~~	1017 ~~412.4~~	906 ~~360.6~~	2.93 ~~6.25~~
3000	207000 ~~83,197~~	1017 ~~412.4~~	906 ~~360.6~~	2.93 ~~6.25~~
4000	207000 ~~83,197~~	1017 ~~412.4~~	906 ~~360.6~~	2.93 ~~6.25~~

Sensitivity of the model predictions to length of dam, L (ft), with cohesive strength, C, set to 100 lb/ft^2, and Manning's n set to 0.06. We chose these values of the other parameters because they are expected to result in larger breach sizes. Notice that when the dam length is limiting (L = 500 ft), BREACH predicts washout of the entire embankment at the top, but predicted breach does not grow wider than the length of the embankment itself. ~~When the dam length is not limiting, model predictions are not sensitive to this parameter.~~

This is incorrect - Model predictions are sensitive if the model is set up correctly (e.g., not constricted by both breach width and tailwater section width.)

L (ft)	Q_p (cfs)	B_t at Peak Flow (ft)	B_b at Peak Flow (ft)	T_p (hr)
	114000	500	412	3.08
500	~~122,550~~	~~500.0~~	~~412.0~~	~~1.81~~
	134000	742	632	5.03
1000	~~128,954~~	~~568.6~~	~~472.2~~	~~2.28~~
~~2000~~	~~128,954~~	~~568.6~~	~~472.2~~	~~2.28~~
~~3000~~	~~128,954~~	~~568.6~~	~~472.2~~	~~2.28~~
~~4000~~	~~128,954~~	~~568.6~~	~~472.2~~	~~2.28~~

- Sensitivity of model predictions to size of tailwater cross-section. The applicant used the tailwater cross-section given in column 2 of the table below (TWCS 0). We investigated the sensitivity of the model by enlarging the tailwater cross-section given by TWCS 1-6 scenarios.

Elevation (ft)	Tailwater Cross-Section (TWCS) Scenarios								
	B_t	n	0	1	2	3	4	5	6
28	0	0.06	0	0	0	0	0	0	0
29	600	0.06	600	1000	1200	1600	2000	2400	3000
32	1200	0.06	1200	1200	1200	1600	2000	2400	3000
34	1600	0.06	1600	1600	1600	1600	2000	2400	3000
40	2000	0.06	2000	2000	2000	2000	2000	2400	3000
45	2400	0.06	2400	2400	2400	2400	2400	2400	3000
50	2800	0.06	2800	2800	2800	2800	2800	2800	3000

The predicted peak discharges, breach widths, and times-to-peak for the TWCS 1-6 scenarios are shown in the table below.

TWCS	Q_p (cfs)	B_t at Peak Flow (ft)	B_b at Peak Flow (ft)	T_p (hr)
0	83,197	412.4	360.6	6.25

Simulation for scenarios 1,2, and 3 are incorrect but were not simulated.

Predictions for 3,4, and 5 were corrected using realistic levee width and tailwater section of 3000 ft.

1	97,314	496.2	385.4	4.43
2	99,203	517.3	406.6	4.76
3	101,493	540.4	429.6	5.05
4	207000 ~~102,971~~	1017 ~~580.1~~	906 ~~469.3~~	2.93 ~~5.70~~
5	207000 ~~102,971~~	1017 ~~580.1~~	906 ~~469.3~~	2.93 ~~5.70~~
6	207000 ~~102,971~~	1017 ~~580.1~~	906 ~~469.3~~	2.93 ~~5.70~~

lection of BREACH scenarios for independent confirmatory analysis

- There is considerable uncertainty in predictions from the BREACH model. There is also considerable uncertainty in predictions from empirical approaches (Wahl 2004). Generally, uncertainty in predictions can be classified as aleatory and epistemic (Merz and Thieken 2005). Aleatory or natural uncertainty arises from inherent variability in natural processes such as randomness of soil properties. Epistemic uncertainty results from incomplete understanding of the natural system being described such as incomplete process description that does not adequately capture the behavior of the system and errors related to measuring parameters used to describe the system.

This statement is incorrect because soil property other than Manning's n is insensitive to breach parameter prediciton.

In the context of this review, the postulated MCR embankment breach occurs due to a piping failure. It has been shown (see FSAR and SER Sections 2.4S.3, 2.4S.4, 2.4S.5, and 2.4S.8) that the MCR would not overtop. The empirical approaches do not explicitly consider the failure mechanism. The BREACH model directly considers the failure mechanism; it can simulate piping and overtopping failures. Therefore, the uncertainty with respect to breach mechanism is reduced because an overtopping-induced failure is eliminated. However, uncertainty still exists in how the BREACH model simulates piping failure of an embankment. Some of this uncertainty arises from incomplete knowledge and understanding of the piping failure process and some from incomplete knowledge of the parameters used in BREACH to simulate piping failures. We have investigated the effect of Z_p, the elevation at which piping failure starts, which can be thought of as partially addressing epistemic uncertainty. Other parameters that are included in the sensitivity analyses described above address aleatory uncertainty.

This portion should be changed as "a piping or landslide failure."

- Based on the sensitivity analyses carried out above, we selected a few parameter sets to run BREACH simulations. Because the BREACH predictions are fairly insensitive to Z_p, C, and Ø, we selected the values of these parameters such that they would generally be expected to result in more conservative peak discharge and time to peak. We used Manning's n of 0.025, 0.050, and 0.075 for our BREACH simulations, because of the high sensitivity of BREACH predictions to Manning's n.

Simulation 1 { Z_p, C, Ø, n }: { 32 ft, 200 lb/ft^2, 15 degrees, 0.025 }
Simulation 2 { Z_p, C, Ø, n }: { 32 ft, 200 lb/ft^2, 15 degrees, 0.050 }
Simulation 3 { Z_p, C, Ø, n }: { 32 ft, 200 lb/ft^2, 15 degrees, 0.075 }

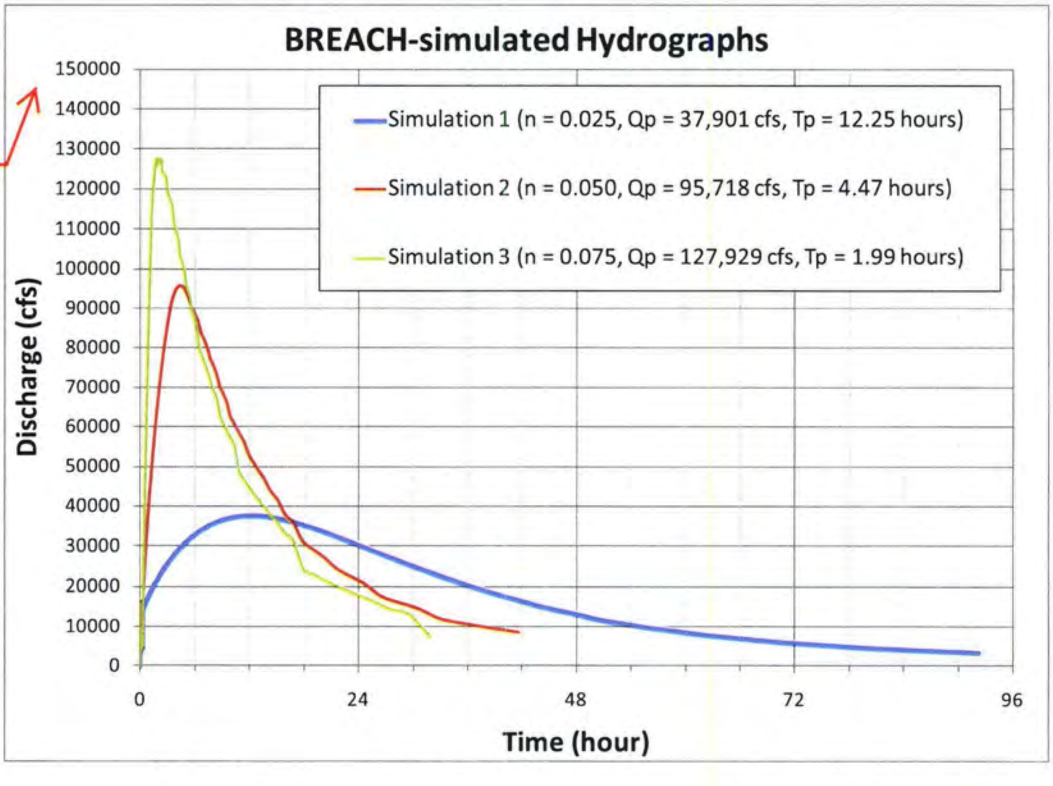
Simulation	Q_p (cfs)	B_t at Peak Flow (ft)	B_b at Peak Flow (ft)	T_p (hr)
1	37,901	263.0	152.3	12.25
2	103000 ~~95,718~~	580 ~~490.6~~	470 ~~379.8~~	5.79 ~~4.47~~
3	194000 ~~127,929~~	989 ~~574.3~~	878 ~~463.6~~	3.29 ~~1.99~~

This run uses n of 0.076 because n of 0.075 gives a BREACH model run-time error.

For simulations shown above, the breach continued to grow after peak flow was reached. For simulation 1, B_t and B_b values at 92 hours were 360.7 and 250.0 ft. For simulation 2, B_t and B_b values at 41.7 hours were 592.4 and 481.7 ft. For simulation 3, B_t and B_b values at 31.9 hours were 743.3 and 632.5 ft.

- **Outflow hydrographs following MCR embankment breach**
  - The hydrographs predicted by BREACH for the three simulations that we used are shown below.

This graph needs to be corrected with correct simulation result.



- **Comparison of MCR embankment breach hydrographs with historical dam breaches with similar characteristics**
  - We used the Reclamation dam breach database to identify historical breaches of dams that have characteristics similar to the MCR. The MCR height of water above the breach, h_w, and volume stored above the breach bottom, V_w, are 21.9 ft (6.68 m), and 152,700 acre-feet (188,352,699 m^3) respectively. The Dam Failure Database was sorted to find entries that were similar to the MCR embankment in terms of h_w

and V_w. The range of h_w considered was 15 to 50 ft (about 4 to 15 m) and the range of V_w considered was 100,000 to 300,000 acre-feet (about 1.23×10^8 m3 to 3.70×10^8 m^3). The database has multiple records for dam break events to allow for different estimates for the same parameter to be included. The sorting treated each record uniquely. The total number of records in the database is 410 but some records are incomplete for some of the parameters. Partial records were retained in the sort.

There were 172 records that has h_w in the range of interest. The minimum and maximum reported breach parameter values are given in the table below:

Parameter	Minimum	Maximum
Water Height above breach bottom (h_w) (m) [ft]	4.05 [13.29]	15.2 [49.87]
Peak flow (Q_p) (m3/s) [cfs]	29.4 [1038]	3115 [110005]
Final breach top width (m) [ft]	9.2 [30.18]	153.0 [502.0]
Final breach bottom width (m) [ft]	1.7 [5.58]	97.0 [318.2]
Average final breach width (m) [ft]	4.7 [15.42]	185.9 [609.9]
Breach formation time (hr)	0.25	1.5
Failure time (hr)	0.5	5.0

The database had nine entries that included volumes above the breach bottom, V_w, in the ranges of interest. Seven of these records are associated with the Teton Dam failure and the remaining two are associated with the Martin Cooling Pond failure.

These historical data could be presented as a reference, however they cannot be used to justify MCR levee breach. It is highly subjective process. Empirical equations are established using several hundred dam breach data, thus we cannot simply ignore the established equations.

Parameter	Teton	Martin Cooling Pond
Water Height above breach bottom (h_w) (m) [ft]	67.06 -83.82 [219.9 -275.0]	8.53 [27.99]
Volume of water above breach bottom (m3) [acre-ft]	3.10×10^8 [251,321]	1.36×10^8 [110,257]
Peak flow (Qp) (m3/s) [cfs]	65,120 - 65,136 [2,299,691 - 2,300,256]	3115 [110,005]
Final breach top width (m) [ft]		
Final breach bottom width (m) [ft]		
Average final breach width (m) [ft]	151 [495]	185 [607]
Breach formation time (hr)	1.25	
Failure time (hr)	4	

The only entries in the database that met both search criteria are associated with the

Martin Cooling Pond.

- **MCR embankment breach parameters estimated from empirical approaches**
  - The following empirical approaches were used. Only those approaches were used that provided estimates of all three parameters – average breach width, failure time, and peak flow.
    - Bureau of Reclamation (1982, 1988)
      - Average breach width (m), $B_{avg} = 3\ h_w$
      - Failure time (hr), $t_f = 0.011\ B_{avg}$
      - Peak flow (m^3/s), $Q_p = 19.1\ h_w^{1.85}$
    - MacDonald and Langridge-Monopolis (1984)
      - Volume of eroded material (m^3), $V_{er} = 0.0261\ (V_w\ h_w)^{0.769}$
      - Failure time (hr), $t_f = 0.0179\ V_{er}^{0.364}$
      - Enveloping peak flow (m^3/s), $Q_p = 3.85\ (V_w\ h_w)^{0.411}$
    - Froehlich (1995a, b)
      - Average breach width (m), $B_{avg} = 0.1803\ K_o\ V_w^{0.32}\ h_b^{0.19}$
      - Failure time (hr), $t_f = 0.00254\ V_w^{0.53}\ h_b^{-0.9}$
      - Peak flow (m^3/s), $Q_p = 0.607\ V_w^{0.295}\ h_w^{1.24}$

We should use peak flow equation (Qp=1.154(Vw*hw)^.44) instead of the bounding equation.

These comparison is not valid because the first 3 are overall breach width while the last 3 are peak breach width.

The table below shows the results of using empirical approaches to predict the characteristics of the MRC embankment failure. We also include the BREACH model predictions for comparison.

Approach	Average Breach Width (m [ft])	Failure Time (hr)	Peak Flow (m^3/s [cfs])
Reclamation	20.0 [65.7]	0.66	5,765 (22,606) ~~4,886 [172,549]~~
MacDonald and Langridge-Monopolis	649.3 [2130.2]	1.67	6152 (217247) ~~21,151 [746,933]~~
Froehlich	127.0 [416.8]	6.98	1,765 [62,315]
BREACH Simulation 1	93.1 [305.4]	12.25	1,073 [37,901]
BREACH Simulation 2	160(525) ~~163.7 [537.1]~~	5.79 ~~4.47~~	2917(103000) ~~2,710 [95,718]~~
BREACH Simulation 3	285(934) ~~209.7 [687.9]~~	3.29 ~~1.99~~	5493(194000) ~~3,622 [127,929]~~

MLM peak flow is similar to the corrected BREACH simulation 3.

Note ~~1:~~ MacDonald and Langridge-Monopolis (1984) approach estimates the volume of material eroded by the breach. The breach width was estimated using the embankment geometry. The embankment geometry shown by Bechtel Energy Corporation (1984, Figure 27) was used to determine the area of cross-section of the MCR embankment. We estimated the volume of eroded material to be 259,479.4 m^3 (9,163,427.4 ft^3). The estimated corresponding average breach width was estimated by dividing the volume of eroded volume by the embankment's cross-section area and is reported above.

Note 2: Froehlich (1995a) equation for failure time uses height of the breach, h_b. The applicant, in response to RAI 2.4.4-14, used h_w in place of h_b in Froehlich's equation for failure time. The applicant's estimate of failure time is 11.1 hr (STPNOC

2010).

Note 3: As reported above, the BREACH model simulations resulted in breach widths that are somewhat larger than those predicted by the Froehlich approach but significantly smaller than the MacDonald and Langridge-Monopolis approach. The failure times in BREACH approach the MacDonald and Langridge-Monopolis prediction for a very conservative parameter set. Predicted peak flows from the BREACH model are generally in the range of that from the Froehlich approach, but significantly smaller than the MacDonald and Langridge-Monopolis approach.

- **A note on application of FLDWAV in estimation of MCR embankment breach outflow hydrograph**
  - The NWS BREACH model produces estimates of the breach growth and breach outflow (hydrograph) over time; these can be coupled to produce sediment flux over time. The growth of the breach is estimated based on hydraulics of the embankment and geotechnical parameters of the embankment material. FLDWAV could be used with prescribed timing parameters that specify breach growth such that FLDWAV-estimated discharge hydrograph and breach formation approximate those produced by BREACH. If the conceptual model for the subsequent flooding includes multiple/cascading breaches or channel networks, FLDWAV would be the appropriate model for simulating the more complex flow scenario. However, in the case of the postulated breach of the MCR embankment, a single breach is the conceptual model and therefore only BREACH is necessary to characterize the outflow event. Therefore, we did not use FLDWAV to estimate the outflow hydrograph from the MCR embankment breach.

NRC FORM 757
NRC MD 10.158
(7-2011)

U.S. NUCLEAR REGULATORY COMMISSION

NON-CONCURRENCE PROCESS

NCP TRACKING NUMBER

TITLE OF SUBJECT DOCUMENT	ADAMS ACCESSION NO.
Re-Analysis of MCR Breach Flood	ML12278A192

SECTION C - TO BE COMPLETED BY DOCUMENT SPONSOR

NAME
Nilesh Chokshi

TITLE
Deputy Director, Division of Site Safety and Environmental Analysis

PHONE NO.
(301) 415-1634

ORGANIZATION
NRO/DSEA

SUMMARY OF ISSUES

See attached DSEA management response (ML12289A775)

ACTIONS TAKEN TO ADDRESS NON-CONCURRENCE

The Division of Site Safety and Environmental Analysis (DSEA) management reviewed each of the primary issues raised by Dr. Hosung Ahn's non-concurrence. To assist DSEA management in it's review of the non-concurrence, six technical experts were requested (by contract with the Office of Nuclear Regulatory Research) to independently review the issues raised by Dr. Ahn and the staff's final safety evaluation report (FSER).

Based on it's review, as informed by the independent contractor reports DSEA management concludes that the staff's safety analysis is technically sound an adequate to conclude that the applicant's hydrology evaluations and analysis as presented in the FSER, comply with all applicable regulations (See ML12289A775).

As part of its revie DSEA management met with Dr. Ahn to discuss his concerns and coordinate to assure management's response accurately represented his concerns.

SIGNATURE--DOCUMENT SPONSOR Nilesh C. Chokshi	TITLE Deputy Director
ORGANIZATION NRO/DSEA	DATE 10/~~15~~ 24/2012
SIGNATURE--NCP REVIEWER Scott Flanders	TITLE Director
ORGANIZATION NRO/DSEA	DATE 10/15/2012

NCP OUTCOME

Non--Concurring Individual: [ ] CONCURS [✓] NON-CONCURS [ ] WITHDRAWS NON-CONCURRENCE (i.e., discontinues process)

AVAILABILTY OF NCP FORM

Non--Concurring Individual: [✓] WANTS NCP FORM PUBLIC [ ] WANTS NCP FORM NON-PUBLIC

[ ] CONTINUED IN SECTION D

SEE SECTION E FOR IMPLEMENTATION GUIDANCE

STP non-concurrence

INTRODUCTION

On June 8, 2011, Dr. Hosung Ahn submitted a non-concurrence report for section 2.4, Hydrologic Engineering of the proposed South Texas Project (STP) Units 3 and 4 Final Safety Evaluation Report (FSER). Dr. Ahn raised concerns with the adequacy of the analysis in FSER section 2.4.4, Potential Dam Failures, section 2.4.5 Probable Maximum Surge and Seiche Flooding, and section 2.4.12 Groundwater. Dr. Ahn's non-concurrence identifies three principle issues:

1. The design basis flood level (DBF) for the proposed site is neither accurate nor conservative because the potential breach size and the resulting outflow of the Main Cooling Reservoir were incorrectly estimated.

2. The applicant's analysis of a combined hurricane and Main Cooling Reservoir breach event is not conservative and the staff's review is inadequate. As a result, use of the Main Cooling Reservoir breach as the DBF event maybe non-conservative.

3. The applicant's proposed maximum groundwater level of 28ft MSL is incorrect, in that during a DBF the staff postulates that the subsurface will be saturated resulting in a groundwater level that is at plant grade (34 MSL). As a result, a departure from the ABWR DCD maximum groundwater level is required.[1] Additionally, the applicant and staff have not considered a saturated subsurface in its structural engineering evaluation, flood protection measures, and PRA analyses in the FSAR and FSER.

As a result of the issues above, Dr. Ahn does not believe the staff's analysis is consistent with staff guidance and the STP application does not meet NRC regulatory requirements. Dr. Ahn's non-concurrence includes an independent analysis of the Main Cooling Reservoir breach and results in a DBF level at the site of 45 ft (13.7 m) MSL, 5 ft (1.5 m) higher than that proposed by STP. Dr. Ahn did not provide independent analyses for the combined events or maximum groundwater issues.

To assist in the review of the non-concurrence, NRO/DSEA requested that the Office of Nuclear Regulatory Research (RES) contract with six technical experts in dam breach analysis and hurricane storm surge. Specifically, the experts were asked to independently evaluate Dr. Ahn's non-concurrence and the staff's FSER and to provide their views on the technical merits of both

DSEA management reviewed each of the three principle issues identified in Dr. Ahn's non-concurrence. Based on these reviews, the DSEA management finds that the staff's analysis is technically sound and adequate to conclude that the applicant's hydrology evaluations and analyses as presented in FSAR section 2.4 complies with all applicable regulations.

In reaching this conclusion, DSEA management reviewed and considered Dr. Ahn's non-concurrence report, the staff's FSER, technical expert reports, and other associated technical documents. The focus of management's review was to assess whether the staff's FSER is technically adequate, consistent with staff guidance, and provides adequate basis to justify the regulatory determination. Dr. Ahn's non-concurrence submittal included a complete analysis of the Main Cooling Reservoir breach. It is clear that Dr. Ahn put forth tremendous effort and we thank him for his hard work and commitment to public health and safety. While

[1] The ABWR DCD requires that the maximum groundwater level is to be no higher than 2 ft (61 cm) below grade surface.

DSEA management maintains the view that the staff's analysis is technically sound and complete, Dr. Ahn's non-concurrence has improved the quality of the FSER. As a result of the issues raised by Dr. Ahn, it was clear that the staff needed to add additional information to the FSER to better support the basis for its conclusions. Experience gained from this process will also be used to decide whether the staff guidance on writing of a SER should be further enhanced to better explain technical basis for staff's decisions.

NON-CONCURRENCE EVALUATION

Summary of Issues

Issue 1

The design basis flood level (DBF) for the proposed site is neither accurate nor conservative because the potential breach size and the resulting outflow of the Main Cooling Reservoir were incorrectly estimated.

Dr. Ahn raises concerns with the applicant's Main Cooling Reservoir breach analysis as well as Pacific Northwest National Laboratory's (PNNL), the NRC staff contractor[2], confirmatory analysis. Dr. Ahn's identified concerns with the empirical method used by the applicant to estimate the size of the Main Cooling Reservoir breach, time to failure, and corresponding outflow as well as the applicant's physically-based numerical method to independently verify the results of its empirical method. Dr. Ahn's principal concerns with the applicant's empirical method are twofold: (1) the applicant selects its breach parameters from multiple methods without adequate bases; and (2) the method used to estimate the Main Cooling Reservoir breach width is not appropriate for the Main Cooling Reservoir and is non-conservative.

Dr. Ahn maintains that the applicant's approach, which used the best fit Froehlich equation to calculate breach width and a bounding MacDonald and Langridge-Monopolis (MLM) equation to calculate the time to failure, is technically unacceptable and results in non-conservative parameters. Dr. Ahn believes the applicant's approach biases the results by using selective methods without adequate justification. Dr. Ahn is of the view that given the high uncertainty in the breach parameters calculated from any accepted empirical method, several methods should be used to predict breach parameters (e.g., breach width, time to failure, and outflow) and uncertainty in each of the methods should be considered. Dr. Ahn references the US Bureau of Reclamation (USBR) and US Army Corps of Engineering (USACE) guidance documents to support his position.

Additionally, Dr. Ahn is concerned with the empirical method the applicant selected to estimate the Main Cooling Reservoir breach width, and the lack of supporting justification for its use.
Dr. Ahn holds the view that the Froehlich equation is not appropriate for estimating the Main Cooling Reservoir breach because the size of the Main Cooling Reservoir breach is greater than 164 ft (50 m). Dr. Ahn cites the US Bureau of Reclamation Dam Breach Manual, specifically Dr. Ahn states that the USBR's dam breach manual (Wahl, 1998) indicates that the Froehlich's equation appears to be the best predictor for the breach widths less than 164 ft because the breach width records used to develop the Froehlich equation is small. Additionally, Dr. Ahn states that the applicant dismisses the applicability of the MLM equation to estimate breach width without justification. Dr. Ahn's independent analysis relies on the MLM equation and other empirical methods to calculate breach parameters he considers appropriately conservative.

[2] PNNL performed work under contract for the NRC staff. The staff accepted PNNL's technical evaluation and used it to prepare the staff's FSER. Hereafter, reference to the staff's analysis includes PNNL's evaluation.

The applicant used the National Weather Service (NWS) BREACH model to independently verify the reasonableness of the breach parameters estimated by the empirical methods.
Dr. Ahn's concerns with the applicant's physically-based method are associated with the input parameters and model setup. Specifically, Dr. Ahn's position is that the roughness coefficient, Manning's n-value, the applicant used in the model is non-conservative. This is of particular importance because the model results are sensitive to this input parameter. Dr. Ahn's view is that the applicant did not appropriately justify the value used in their model and further did not adequately consider all Main Cooling Reservoir materials such as sand chimney and cement blocks when selecting the Manning's n-value. Additionally, the applicant's model does not consider erosion below grade level of the Main Cooling Reservoir (29 ft MSL). Dr. Ahn maintains consideration of erosion resulting from scouring is important and will affect the resulting breach outflow estimate. Dr. Ahn also maintains that the applicant did not properly set up the model because they limited the tailwater cross-section width to 600 ft (182.8 m) and the crest width to 1000 ft (304.8 m). Limiting the tailwater and crest cross-sections limits the breach width. Dr. Ahn's view is that the downstream tailwater condition of the Main Cooling Reservoir breach is a wide two-dimensional overland flow and the tailwater cross-section and crest cross-section should reflect these conditions.

Dr. Ahn also raises concerns about PNNL's confirmatory analysis that are similar to those raised about the applicant's analysis. Specifically, he believes the analysis is inadequate in that it excludes the MLM method without explanation and NWS BREACH input parameters and model setup are incorrect. Additionally, he raises concern with PNNL's comparative analysis, which uses a 1979 breach event of the Martin Cooling Pond in Florida to conclude that the Main Cooling Reservoir breach parameters and resulting STP site flood levels are reasonable. He also maintains that PNNL inappropriately relied solely on the NWS BREACH model results for
their confirmatory analysis, which is inconsistent with industry practice. Dr. Ahn states that industry experts, including the developer of the NWS BREACH model, recommend the model be used only to supplement breach analyses.

Issue 2

The applicant's analysis of a combined hurricane and Main Cooling Reservoir breach event is incorrect and the staff's review is inadequate. As a result, use of the Main Cooling Reservoir breach as the DBF event maybe non-conservative.

Dr. Ahn maintains that the applicant's probable maximum hurricane (PMH) and resulting surge estimate are not conservative because the PMH is based on a 1979 National Oceanic and Atmospheric Administration (NOAA) manual (NWS-23) that has not been updated to include storms from the past 30 years. Dr. Ahn's states that it is generally known that the intensity and frequency of hurricanes have increased since the document was issued. Further, he is of the view that the staff's contractor, PNNL, did not adequately review the hurricane storm surge. Specifically, he states staff did not review the input and outputs of the applicant's ADCIRC model or performed any confirmatory analysis, other than SLOSH simulations. Dr. Ahn states that the applicant discarded the SLOSH model because it was less accurate than the ADCIRC model. It is Dr. Ahn's opinion that NRC should review the applicant's ADCIRC model to confirm the adequacy of the surge estimates. Without doing so, the staff has no basis for concluding that the combined hurricane and Main Cooling Reservoir breach is not a credible event.

Issue 3

The Maximum Groundwater level of 28 ft MSL is incorrect, in that during a DBF the subsurface will be saturated resulting in a groundwater level that is at plant grade (34 ft MSL). As a result, a departure from the

ABWR DCD maximum groundwater level is required.[3] Additionally, the staff has not considered a saturated subsurface in its structural engineering evaluation in section 3.8 of the FSER.

Dr. Ahn is of the view that the applicant needs a departure from the ABWR DCD requirement that the maximum groundwater level must be 2 ft (61 cm) below grade. His position is based on the fact that the ABWR DCD maximum ground water level site parameter will be exceeded during a design basis flood. Dr. Ahn points out that the staff's groundwater analysis conservatively assumes that the stone layer and clay cap STP proposes to minimize groundwater infiltration is eroded away during a DBF; however, the applicant and staff does not provided adequate basis for their assumptions and the corresponding structural, flood protection, and PRA analyses do not appropriately take into account the effect of having saturated conditions at the plant grade level.

Evaluation

To evaluate Dr. Ahn's non-concurrence, DSEA management considered several sources of information, including scientific journals, federal and state agency guidance documents, NRC regulations, and NRC staff guidance. Additionally, at the request of NRO, the RES contracted with six technical experts to review Dr. Ahn's non-concurrence and the staff's FSER. These technical reports were also used by DSEA management in evaluating the non-concurrence.

<u>Issue 1 Main Cooling Reservoir Breach</u>

Section 2.4.4 of the staff's FSER evaluates the potential failure of onsite, upstream, and downstream water-control structures, such as dams. The purpose of the analysis is to ensure safety related system, structures and components are protected from potential external hazards. Section 2.4.4 considers potential site flood level from two postulated dam failures scenarios: failure of all dams on the Colorado River upstream of the STP site[4] and breach of the Main Cooling Reservoir. The applicant and the staff determined that a breach of the Main Cooling Reservoir is the controlling dam failure flood event for the STP site.

<u>Applicable Regulations</u>

- 10 CFR Part 100, as it relates to identifying and evaluating hydrological features of the site. The requirement to consider physical site characteristics in site evaluations is specified in 10 CFR 100.20(c).
- 10 CFR 100.23(d)(3), as it sets forth the criteria to determine the siting factors for plant design bases with respect to seismically induced floods and water waves at the site.
- 10 CFR 52.79(a)(1)(iii), as it relates to identifying hydrologic site characteristics with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

<u>NRC staff guidance</u>

- Section 2.4.4 of NUREG–0800

[3] The ABWR DCD requires that a site's maximum groundwater level is to be no higher than 2ft (61 cm) below grade surface.
[4] The applicant considers two permutations of how the dams on the Colorado River fail. Both permutations consider the same flood event, however, the different assumptions on how the dams fail result in changes in the arrival time and peak downstream discharge.

- RG 1.206, "Combined License Applications for Nuclear Power Plants (LWR Edition)"
- RG 1.27, "Ultimate Heat Sink for Nuclear Power Plants"
- RG 1.59, "Design Basis Floods for Nuclear Power Plants," as supplemented by best current practices
- RG 1.102, "Flood Protection for Nuclear Power Plants"

Issue(s)

Dr. Ahn raises several concerns with the applicant's Main Cooling Reservoir breach analysis and the staff's confirmatory analysis. The applicant estimated the Main Cooling Reservoir breach using empirical equations to estimate breach parameters such as breach width and time-to-peak discharge; the FLDWAV model to estimate flood discharges; and the two-dimensional model RMA2 to estimate the spread of the flood flow downstream of the breach and the corresponding flood elevation at the site. The applicant then used NWS BREACH, a physically based model, to independently check the dam breach parameters and the FLDWAV discharge values. The staff used NWS BREACH to verify the adequacy of the applicant's Main Cooling Reservoir breach analysis and performed a Comparative Analysis, which compared the results of their (the staff's) confirmatory NWS BREACH analysis with data from actual dam failures. The staff also reviewed the adequacy of the applicant's empirical methods and its RMA2 model. Dr. Ahn raises several concerns with the applicant's analysis both regarding the empirical methods and NWS BREACH model. He also raises concerns with the staff's review. While there are many aspects of each analysis Dr. Ahn cites concern with, which will be discussed in more detail later, he fundamentally believes:

1. The empirical method used by the applicant is non-conservative, in that the equation selected to calculate the breach width is not appropriate for the Main Cooling Reservoir and the equation's prediction uncertainty was not appropriately accounted for.
2. The applicant and NRC did not properly set up the NWS BREACH model in that it used non-conservative parameters that limited the size of the Main Cooling Reservoir breach and corresponding outflow.

Assessment

After review of Dr. Ahn's non-concurrence report, the staff's FSER, technical expert reports, and other associated technical documents, DSEA management concludes that the Main Cooling Reservoir breach analysis by the applicant is based on realistically conservative assumptions and the staff conducted a sufficient evaluation to determine if the applicant's analysis was technically sound and complied with all regulatory requirements. Dr. Ahn's analysis was thorough and highlighted the importance of certain assumptions and parameters associated with an embankment breach analysis. While it is clear that expert engineering judgment is necessary in an embankment breach analysis, there are certain industry practices that guide these analyses. In an effort to evaluate the issues raised in Dr. Ahn's non-concurrence, DSEA management relied on these industry practices as a guide.

Enclosure 1 summarizes assumptions and results of the Main Cooling Reservoir breach analyses prepared by the applicant, the staff, and Dr. Ahn. From review of the table in Enclosure 1 and consideration of the key factors that influence the flood level at site, there are a few key factors that warrant close examination in evaluating Dr. Ahn's non-concurrence. In the Enclosure 1 table, the rows highlighted in red represent key parameters or results that DSEA
management considers as central to determining the DBF and that are in dispute. Rows highlighted in yellow are other areas that are also in dispute, but are not as critical to the results as the areas highlighted in red.

DSEA management will discuss the highlighted issues associated with the empirical analysis, followed by those associated with the physically-based model.

Empirical Analysis

Dr. Ahn maintains that the applicant's approach of using only the best fit Froehlich equation, to calculate breach width and a bounding MacDonald and Langridge-Monopolis (MLM) equation to calculate the time to failure is technically unacceptable and results in non-conservative parameters. Dr. Ahn believes the applicant's approach biases the results by using selective methods without adequate justification. Dr. Ahn is of the view that given the high uncertainty in the breach parameters calculated from any accepted empirical method, several methods should be used to predict breach parameters (e.g., breach width, time to failure, and outflow) and the prediction uncertainty associate with each of the methods should be considered. Dr. Ahn references guidance documents by the US Bureau of Reclamation (USBR) and US Army Corps of Engineering (USACE) to support his position.

Additionally, Dr. Ahn holds the view that the Froehlich equation is not appropriate for estimating the Main Cooling Reservoir breach because the size of the Main Cooling Reservoir breach is greater than 164 ft (50m). Dr. Ahn cites the US Bureau of Reclamation Dam Breach Manual, specifically Dr. Ahn states that the USBR's dam breach manual (Wahl, 1998) indicates that the Froehlich's equation appears to be the best predictor for the breach widths less than 164 ft because the breach width records used to develop [for] the Froehlich equation is small. Additionally, Dr. Ahn states that the applicant dismisses the applicability of the MLM equation to estimate breach width without justification. Dr. Ahn's independent analysis relies on the MLM equation and other empirical methods to calculate breach parameters he considers appropriately conservative.

To evaluate this issue, DSEA management first turned to the technical expert reports prepared in support of the non-concurrence evaluation. The staff, through RES, contracted with six independent technical experts to evaluate the non-concurrence. Three of the six (Dr. Anthony Wahl, Dr. Robert Patev, and Prof. Gregory Baecher) focused on the Main Cooling Reservoir breach analysis. All three of the experts indicated that use of the Froehlich equation to estimate breach width is appropriate and the best choice. Wahl makes two points in his discussion regarding the use of the Froehlich equation. First he points out that the statement attributed to his 1998 report regarding the applicability of the Froehlich equation for breach widths greater than 164 ft is a misinterpretation. Specifically he notes that it is not correct to conclude "Froehlich is inappropriate for breach widths greater than 164 ft just because it is the better choice for breach widths less than 164ft." Dr. Wahl also notes that based on his 2004 report, the uncertainty of breach width predictions using Froehlich equation is half that of the predictions using the MLM equation. Dr. Patev notes that the construction of the Main Cooling Reservoir is such that a breach would likely evolve slowly thus would further support the use of the Froehlich equation. Baecher notes that the "MLM model provides a poorer fit to historical data than does the Froehlich model and thus seems an inferior choice by a substantial margin."

DSEA management also considered guidance provided in several documents, including the state of Colorado Guidelines for Dam Breach Analysis. That document reports that "Froehlich method is recommended for small or large dams with a volume greater than 100 AF, as it yields conservative, but reasonable results." The report also states that the MLM method may be appropriate in certain situations. The report then refers the reader to a table that provides guidance on which method to use depending upon the storage intensity of the dam, where storage intensity is defined as the storage volume divided by the maximum depth of water behind the breach ($SI=V_w/h_w$). In the case of the Main Cooling Reservoir, the SI is 6,972 (152,700 ac-ft/21.9 ft), significantly higher than the 21 that would have the Main Cooling Reservoir classified as a high intensity dam. In this situation, it is recommended that Froehlich or possibly the MLM equation be used. The state of Colorado guidance also notes that engineering judgment is needed with the predicted parameters and provides a few tools to assist the user. One tool is the ratio of the linear erosion rate (ER) to maximum depth of water behind the dam prior to breach (h_w). The linear erosion rate is the average breach width divided by the time to failure (B_{avg}/T_f). Using the MLM method the E_R/h_w is 58.2[5]. The Froehlich value is 2.72. The guidance states that if the E_R/h_w is less than 1.6, then either the T_f is too long or the B_{avg} is too small, and if the E_R/h_w is greater than 21 then the parameters are suspect.

Considering the information provided by the experts and that gleaned from the state of Colorado guidance, DSEA management concludes the use of the Froehlich is an appropriate method to use to estimate the Main Cooling Reservoir breach width.

With regards to estimate the time to failure, all three of the experts also indicate that the use of MLM equation is appropriate and conservative. Using the Froehlich method, the time to failure is estimated to be 6.98 hours, which is less conservative than the 1.7 hours used by the applicant based on the MLM method.

Physically Based Modeling

Dr. Ahn's concerns with the applicant's NWS BREACH model are associated with the input parameters and model setup. Specifically, Dr. Ahn's position is that the roughness coefficient, Manning's n-value, the applicant used in the model is non-conservative. This is of particular importance because the model results are sensitive to this input parameter. Dr. Ahn's view is that the applicant did not appropriately justify the value used in their model and further did not adequately consider all Main Cooling Reservoir materials such as sand chimney and cement blocks when selecting the Manning's n-value. Additionally, the applicant's model does not consider erosion below grade level of the Main Cooling Reservoir (29 ft MSL). Dr. Ahn maintains consideration of erosion resulting from scouring is important and will affect the resulting breach outflow estimates. Dr. Ahn also maintains that the applicant did not properly set up the model because they limited the tailwater cross-section width to 600 ft (182.8 m) and the crest width to 1000 ft (304.8 m). Limiting the widths of the tailwater and crest cross-sections limits the breach width. Dr. Ahn's view is that the downstream tailwater condition of the Main Cooling Reservoir breach is a wide two dimensional overland flow and the tailwater cross-section and crest cross-section should reflect these conditions.

The Manning's n value is a measure of the hydraulic roughness, that is, the amount of resistance water experiences when passing over land and channel features, or the resistance when flowing through a conduit or pipe. In this case, the Manning's n value is an estimate of the resistance of flow through the breach. Typically, Manning's n values in these types of analysis can range from 0 (frictionless) to 0.1 (very rough).[6] The NWS BREACH model allows the user to input the Manning's n value or to input the median grain size of the embankment material and allow the model to calculate the Manning's n value using the Strickler equation based on the median grain size. The Manning's n value for the flow through the breach can have a significant effect on the breach width and peak discharge. The Manning's value incorporates the effects of both discharge through the breach width and erosion that expand the breach width.

In the FSER, the staff determined that the NWS BREACH model estimates larger peak flows for larger Manning's n values. The staff also explains that use of the Strickler equation would give unreasonably low (i.e., 0.0001-0.004) Manning's n values when using the median grain size for the Main Cooling Reservoir. Therefore, the staff varied the Manning's n value from 0.001 to 0.08 to conservatively cover extreme ranges of

[5] Dr. Ahn calculates a breach width of 1736 ft using MLM equation. Staff and DSEA management calculated a value of 2130 ft. Using Dr. Ahn's value, the value of E_R/h_w is 47.5.
[6] Chow provides manning n values greater than 0.1, however, there are very few values exceeding 0.1 and are typically associated with channels having significant vegetation (medium to dense). These situations are clearly not applicable for estimating the manning's n for the MCR breach.

this parameter. While the staff initially set the upper end of the range to 0.08, it noted that the Manning's n value is an estimate of the roughness of the embankment material only, other considerations such as vegetation, channel meanders, and other features should not be considered. The staff noted in the FSER that based on the literature only bare earth materials should be considered for the Main Cooling Reservoir embankment Manning's n value. The literature suggest Manning's n values for bare earth materials range from 0.012 (for flow over fine sand or concrete) to 0.07 (for flow over boulders). Therefore the staff concluded that a value of 0.08 was unrealistically high. The staff did use a Manning's n value of 0.075 for its NWS BREACH confirmatory analysis, and the applicant used 0.05.

Only one of the three experts spoke directly to the concerns raised about the Manning's n value for the Main Cooling Reservoir. Dr. Wahl indicated that for the Main Cooling Reservoir embankment a Manning's n value of 0.025 was reasonable and conservative, a value 0.05 seems very conservative, and a value of 0.075 does not seem credible. Dr. Wahl also addresses Dr. Ahn's concern that sand chimney and cement blocks were not considered in selecting the Manning's n value. He expressed the view that the method used by Dr. Ahn to calculate the Manning's n value was not applicable for the Main Cooling Reservoir and consideration given to some factors, such as the sand chimney and the cement blocks would realistically not have a bearing on the Manning's n value. Dr. Wahl noted that Dr. Fread, the developer of NWS BREACH, intended for the Manning's n value for the flow through the breach to be related to the embankment materials only. This is because the highly energetic dam breach outflows have such power associated with them that they forcibly remove any such features and hence have little effect on the flow.

The staff and Dr. Ahn uses similar Manning's n values in their breach analyses. The applicant produced breach peak discharge estimates similar to that of the staff. DSEA management concludes the Manning's n values used by the applicant and staff to characterize embankment material yielded appropriately conservative peak discharge estimates.

Dr. Ahn questions the applicant's and the staff's decision not to consider scour under the Main Cooling Reservoir in their respective NWS BREACH analyses, and the applicant's and the staff's assumptions for the tailwater cross section. Dr. Ahn's makes different assumptions for the scour hole and tailwater cross section. The applicant estimated that a scour hole with dimensions of 20 ft depth, 203 ft length, and 380 ft width would form downstream of the toe of the Main Cooling Reservoir. The staff concluded that based on geotechnical information regarding Main Cooling Reservoir embankment foundation soils, the formation of a scour hole immediately below the main cooling reservoir embankment would be unlikely. Therefore, the staff agrees with the applicant's conclusion that the scour hole formation due to the postulated main cooling reservoir embankment breach would more likely occur downstream of the embankment in native uncompacted soil areas than in the compacted soils adjacent to on underlying the embankment.

All three of the experts provide similar views on the size and location of scouring hole. Patev indicates that the geotechnical characteristics of the Main Cooling Reservoir foundation, which is engineered backfill consisting of high strength clays, make it unlikely that a scour hole will form during a dam breach event. Baecher states that while the staff should have given more consideration to this issue, the in-situ geotechnical conditions of the foundation, while not eliminating the possibility of scour, certainly mitigate scour. Dr. Wahl expresses the view that Dr. Ahn does not clearly explain how the scour hole was included in the model and how physically the scour hole increases the outflow. Therefore, the results of the scour sensitivity model must be discounted.

Dr. Ahn also indicated that the tailwater cross section values selected by the applicant inappropriately limited the breach width. The applicant and the staff used a tail water section of 600 ft at the bottom of the breach progressing to 2800 at elevation 50 ft MSL. The staff's sensitivity analysis did not suggest that tailwater cross-section was a dominant factor in development of conservative estimates for breach parameters. For the three cases where the asymptotic limit was reached, the breach width did not attain the full tailwater cross-sectional width initially specified in NWS BREACH model inputs indicating the absence of artificial constraints on breach development.

DSEA management looks to the analysis provided by Dr. Wahl to reconcile the differing views between the staff and Dr. Ahn. Dr. Wahl makes several instructive points. First, he notes that the primary source of the increase in breach width and outflows associated with Dr. Ahn's analysis is the large Manning's n value. He points out that with such a high value of Manning's n the sensitivity to the tailwater section is minor. He also notes that the staff's sensitivity analysis was conducted with a Manning's n value of 0.05 and should have been done at 0.075 since the staff relied on that value in their final run. Dr. Wahl states that the NWS BREACH model is not well equipped to estimate the tailwater conditions for the Main Cooling Reservoir because water exiting the breach flows directly into a large open plain, not a river channel. Dr. Wahl states that to effectively model such conditions a two dimensional model should be used. Dr. Wahl states "since NWS BREACH is incapable of 2D modeling of the tailwater, the tailwater cross section should be sized to represent a realistic flow spread." He then concludes that the staff's model provides a realistic tailwater cross section. His view is that Dr. Ahn's tailwater section would require an unrealistic flow by requiring the water immediately exiting the breach to spread broadly and thinly over a 3000 ft-wide section.

Staff Review Methodology

Dr. Ahn raises concern with PNNL's Comparative Analysis, which uses a 1979 breach event of the Martin Cooling Pond in Florida to conclude that the Main Cooling Reservoir breach parameters and resulting STP site flood levels are reasonable. He also maintains that PNNL inappropriately relied solely on the NWS BREACH model results for their confirmatory analysis, which is inconsistent with industry practice. Dr. Ahn states that industry experts, including the developer of the NWS BREACH model, recommend the model be used only to supplement breach analyses.

The staff compared the predictions of peak discharge from the NWS BREACH model to historical observations of dam breaches compiled by Wahl (1998). The staff's motivation for conducting a comparative analysis with historical breaches was to provide an additional confirmation that the conservative physical model simulations were realistic. The State of Colorado uses a similar approach to estimate dam breach parameters (State of Colorado, 2010). Using the Wahl (1998) database, the staff identified historical breaches of dams that have characteristics similar to those of the main cooling reservoir.

As noted by the staff and Dr. Wahl, Comparative Analysis to confirm results from other methods has been used by others. While Baecher questioned the comparisons between the Main Cooling Reservoir and the other dam failures considered by the staff, he does not question the acceptability of the method. DSEA management believes the staff's use of comparative analysis is appropriate. Further, DSEA management believes the staff's approach to use the NWS BREACH model to conduct a confirmatory review was appropriate. The staff's role is to review the applicant's technical analysis used to demonstrate compliance with NRC regulations. Such a review does not require the staff to conduct a detail analysis consistent with industry standards, but instead to assure that the applicant's technical analysis is sound and appropriately complies with NRC guidance and industry practices. It should also be noted that the staff did review all aspects of the applicant's breach analysis, including the empirical methods and FLDWAV.

Dr. Ahn raised several concerns regarding the Main Cooling Reservoir breach analysis and conducted an extensive independent analysis that showed a DBF elevation significantly higher than the staff's. In evaluating the issues raised, DSEA management focused on the adequacy of the staff's review of the applicant's analysis. The focus of the staff's review is to assure that the applicant made appropriate assumptions and that the applicant has demonstrated compliance with the regulations and the staff has reasonable assurance that the facility can be constructed and operated in a manner protective of public health and safety. In this case, DSEA management concludes that the staff's review of the applicant's analysis is technically sound, appropriately conservative, and compliant with NRC regulations.

FSER Changes

The staff revised portions of 2.4.4 to add additional basis for its conclusions. Additional text was added to explain the staff's review of the applicant's empirical methods. Additional discussion was added to explain the tailwater sensitivity analysis.

Issue 2 Combine Hurricane and Main Cooling Reservoir Breach

Section 2.4.5 evaluates the probable maximum storm surge and seiche. The purpose of the analysis is to ensure safety related system structures and components can withstand the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

Applicable Regulations

- 10 CFR Part 100, as it relates to identifying and evaluating hydrological features of the site. The requirement to consider physical site characteristics in site evaluations is specified in 10 CFR 100.20(c).
- 10 CFR 100.23(d)(3), as it sets forth the criteria to determine the siting factors for plant design bases with respect to water levels and wave action at the site.
- 10 CFR 52.79(a)(1)(iii), as it relates to identifying hydrologic site characteristics with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

NRC guidance

- Section 2.4.5 of NUREG 0800
- RG 1.206, "Combined License Applications for Nuclear Power Plants (LWR Edition)"
- RG 1.27, "Ultimate Heat Sink for Nuclear Power Plants"
- RG 1.59, "Design Basis Floods for Nuclear Power Plants," as supplemented by best current practices
- RG 1.102, "Flood Protection for Nuclear Power Plants"

Issue

Dr. Ahn raises two main issues regarding the staff's consideration of a combined hurricane and Main Cooling Reservoir breach. The first is that the staff's probable maximum hurricane, for which the probable maximum storm surge (PMSS) is estimated, is not conservative because it is based on NOAA's NWS-23 report that is more than 30 years old. Secondly, Dr. Ahn states that staff cannot be certain of the adequacy of the applicant's estimated probable maximum storm surge without reviewing the applicant's ADCIRC model. As a result of these two issues, he asserts that the staff's consideration and evaluation of a combined event involving a hurricane and Main Cooling Reservoir breach is incorrect.

Assessment

After review of Dr. Ahn's non-concurrence report, the staff's FSER, technical expert reports, and other associated technical documents, DSEA management concludes that the probable maximum hurricane (PMH) and corresponding PMSS are appropriately conservative to demonstrate compliance with all applicable regulations. Likewise, DSEA management concludes that the staff appropriately reviewed the applicant's ADCIRC model. With these two conclusions, it is DSEA management's view that the combined hurricane and Main Cooling Reservoir breach event was appropriately considered. We do agree with Dr. Ahn's position that consideration of more recent storm data is necessary in supporting the basis that the PMH is appropriately conservative. To this end, Dr. Ahn's non-concurrence has identified the need to revise the FSER to provide additional technical basis to support the staff's conclusion regarding the PMH and PMSS. The discussion below provides the basis for this decision.

Discussion

In section 2.4.5 of the FSER, the staff evaluates the applicant's proposed PMH, including the methodology used to determine the PMH. The applicant used NOAA NWS-23 report to determine the PMH. The development of NWS-23 started in 1975 sponsored by the NRC and the Army Corps of Engineers. The purpose of the study was to develop revised values of meteorological parameters for wind fields to estimate the Standard Project Hurricane and the Probable Maximum Hurricane for various locations along the Gulf and East coasts from Texas to Maine. In addition to developing meteorological parameters, NWS-23 provides guidance on how to use the parameters to develop the PMH. According to NWS-23, a PMH is defined as a "hypothetical steady state hurricane having a combination of values of meteorological parameters that will give the highest sustained wind speed that can probably occur at a specified coastal location." The report also states that meteorological parameters could be used to estimate the probable maximum storm surge (PMSS) at specific coastal locations when the storm approaches along the critical track. It is important to note that NWS-23 report states that hurricane and typhoon data through 1974 and "an understanding of hurricane behavior through 1977" was used to develop the meteorological parameters and methodology to estimate the PMH.

The staff prepared a confirmatory calculation to determine if the applicant appropriately used NWS-23 to develop the PMH. The table below provides a comparison of certain parameters of the staff's calculated PMH and the applicant's calculated PMH.

Comparison of Staff and Applicant PMH

Parameter (units)	Staff Value	Applicant
Central pressure $P_{o=} P_w - \Delta P$ (cm / in. Hg)	66.52 / 26.19	66.52 / 26.19
ΔP (cm / in. Hg)	9.98 / 3.93	9.98 / 3.93
Peripheral pressure P_w (cm / in. Hg)	76.5 / 30.12	76.50 / 30.12
Radius of maximum winds R (mi)	8-33.8 / 5-21	8-33.8/ 5 to 21
Forward speed T (m/s / knot)	3.1-10.3 / 6-20	3.1-10.3 / 6 to 20
Stationary hurricane gradient velocity (m/s / mph)	66.92 / 149.7	68.0 to 71.5 m/s (152 to 160 mph).

The staff concluded that the applicant appropriately used NWS-23 to develop the PMH. In addition to calculating the PMH using NWS-23, the staff issued a request for additional information (RAI) that asked the applicant whether "effort was made to adjust the estimated PMH parameters in light of more recent hurricanes that have occurred since the publication of the NOAA NWS 23 report." The applicant's response indicated that a recent NOAA report indicated that the period from 1945-1970 was the most active period in recent decades. Therefore, the applicant stated that because NWS-23 included this period, the PMH calculated is appropriately conservative and will account for any future variability in the climate. The applicant did not provide any specific information that indicated they independently compared the parameters of storms since 1978 to those of the PMH, and the staff's FSER did not include any discussion beyond a summary of the RAI and the applicant's response.

Although the PMH is based on data until the 1970's, it is not based on an individual historical storm, but instead is developed from a set of meteorological parameters that results in a hypothetical hurricane that is expected to result in the highest sustained winds that may occur at a specific coastal location. Therefore, DSEA management does not agree that one can simply conclude that the PMH is non-conservative because it does not consider more recent data. However, we do agree that the decision on the adequacy of the PMH must be informed by consideration of more recent storms.

To facilitate review of Dr. Ahn's non-concurrence, six independent experts were consulted to review the non-concurrence and the staff's FSER. Three of the six (Dr. Jennifer Irish, Dr. Donald Resio and Dr. Rick Luettich) focused on the PMH and PMSS analysis. All three of the experts discussed all or some of the six parameters listed below in their assessment of the adequacy of the PMH and PMSS.

(1) Storm Intensity (as a function of ΔP: Peripheral pressure(P_w)- Central Pressure (P_o))
(2) Storm Size as represented by the radius of maximum winds (RMW)
(3) Storm forward speed
(4) Storm track angle (relative to the coast)
(5) Landfall location
(6) Holland B parameter (peakedness of wind speed distribution within the storm)

Each of the experts spoke to the importance of all or some of these parameters on the PMSS. There were a few parameters that received the most attention. All three of the experts emphasized the importance of storm size in assessing storm surge. It was noted that a large, slow moving, less intense (e.g., Category 3) storm could generate a larger storm surge than a very intense (e.g., Category 5), small, fast moving storm. All three of the experts concluded that the applicant's PMH RMW (21 nmi) is not representative of the upper bound of Gulf of Mexico hurricanes that made landfall. Dr. Resio indicated that the RMW is only slightly larger than the median value and cannot be considered a conservative value. Dr. Irish and Dr. Luettich noted that the NWS-23 database does not reflect consideration of recent large hurricanes such as Ike, Rita, and Katrina. Dr. Resio concluded that while the storm size is not conservative, other parameters of the PMH are sufficiently conservative to offset the lack of conservatism in the assumed storm size. Thus, he concluded that the PMH was conservative even in light of more recent data than that used in NWS-23. The other experts concluded that the staff should conduct some sensitivity study to compare PMSS resulting from larger, less intense storms that would be representative of storms that occurred in the Gulf of Mexico since NWS-23 was written. The staff also investigated how the PMH compared to more recent storms. The NRC staff determined that there were fifty-four hurricanes that impacted Texas between 1851 and 2008 with18.5 percent occurring outside the NWS-23 reporting period. No hurricane greater than Category 4 has ever made landfall in Texas and all Category 4 hurricanes impacting Texas occurred within the NWS-23 reporting period. For the United States, only 17 percent of the total hurricanes that impacted the U.S. occurred after the NWS-23 reporting period. Looking at the twelve most intense hurricanes to hit the U.S., only three occurred outside of the NWS-23 reporting period. Therefore, the staff determined that the applicant's use of NOAA NWS Report 23 (NOAA, 1979) in deriving the PMH is reasonable and conservative.

While the staff's guidance does not require applicants to use the ADCIRC model, industry experts accept this as the state-of-the-art storm surge modeling tool. Dr. Ahn expressed concern that the staff did not review the input and outputs of the applicant's ADCIRC model or performed any confirmatory analysis, other than SLOSH simulations. In reviewing the FSER, the staff did discuss the applicant's ADCIRC model. Specifically, the staff discusses in the FSER its review of the input parameters, model setup, and results. The staff concluded that the input parameters, model set up, and results were appropriate. The applicant's ADCIRC model was also reviewed by the industry experts. While each expert expressed some concerns with applicant's setup of the ADCIRC model, in general their concerns can be explained by considering what the staff deems as an acceptable approach to estimating the PMSS. For example, one of the experts expressed concern that the applicant used more realistic description of small-scale land characteristics for the ADCIRC model than used in the SLOSH and SURGE model. Such an approach is consistent with the staff's guidance to allow applicant's to employ a hierarchical approach. Specifically, this approach allows applicants to start with very conservative assumptions and then add more realism until arriving at a realistically conservative set of assumptions. The staff specifically reviewed and discussed in the FSER, the applicant's assumptions and bases for the more realistic small-scale land characteristics assumptions.

FSER Changes

Since the issuance of Dr. Ahn non-concurrence, the staff has supplemented the FSER to provide additional basis to justify its conclusion that the PMH proposed by the applicant is appropriately conservative. Specifically, the staff modified the FSER sections 2.4S.5.4.1 and 2.4S.5.6 to include a discussion of more recent storms and how they compare to the PMH. FSER section 2.4S.5.4.2 was modified to add a sensitivity study that compared PMSS resulting from larger, less intense storms to the PMSS resulting from the PMH.

Issue 3 Maximum Groundwater level

In Section 2.4.12, the applicant addresses groundwater conditions in terms of influences on structures and water supply.

Applicable Regulations

- 10 CFR Part 100, as it relates to identifying and evaluating hydrological features of the site. The requirement to consider physical site characteristics in site evaluations is specified in 10 CFR 100.20(c).

- 10 CFR 100.23(d)(3), as it sets forth the criteria to determine the siting factors for plant design bases with respect to seismically induced floods and water waves at the site.

- 10 CFR 52.79(a)(1)(iii), as it relates to identifying hydrologic site characteristics with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

NRC staff guidance

- Section 2.4.12 of NUREG 0800

- RG 1.206, "Combined License Applications for Nuclear Power Plants (LWR Edition)"

Issue(s)

Dr. Ahn is of the view that the applicant needs a departure from the ABWR DCD requirement that the maximum groundwater level must be 2 ft (61 cm) below grade. His position is based on the fact that the ABWR DCD maximum ground water level site parameter will be exceeded during a design basis flood. Dr. Ahn points out that the staff's groundwater analysis conservatively assumes that the stone layer and clay cap STP proposes to minimize groundwater infiltration is eroded away during a DBF; however, the applicant and staff does not provided adequate basis for their assumptions and the corresponding structural, flood protection, and PRA analyses do not appropriately taken into account the effect of having saturated conditions at the plant grade level.

Assessment

After review of Dr. Ahn's non-concurrence report and the staff's FSER, DSEA management concludes that all necessary departures have been requested and that the staff's groundwater analysis and corresponding structural, PRA, and flood protection analysis appropriately consider ground water conditions for all conditions including the DBF.

The applicant adopts a site characteristic for a maximum groundwater level of 8.5 m (28 ft) MSL based on field measurements and modeled post-construction conditions. The applicant states that the post-construction plant grade will be approximately 10.4 m (34 ft) MSL. According to the DCD requirement (i.e., maximum groundwater level is to be greater than 61 cm [2 ft] Below Ground Surface), the maximum groundwater level shall be no higher than 9.75 m (32 ft) MSL. The applicant evaluates hydrostatic loading by comparing two calculations of hydrostatic load that are (1) based on the DCD requirement, and (2) based on the site characteristic. The applicant states that the site characteristic of 8.5 m (28 ft) MSL satisfies the DCD requirement of 61 cm (2 ft) below plant grade and exhibits a satisfactory hydrostatic pressure.

As discussed in FSAR Section 3.8, the DCD site parameter "maximum groundwater level" has been used in conjunction with the most required load combinations, including normal loads and the combination with earthquake loads, the combination with severe winds, and the combination with tornado loads. The DCD site parameter "maximum flood level" is set at 30.5 cm (1 ft) below grade level. This would indicate that the design expected for the maximum flood level to exceed the maximum groundwater level. The maximum flood level at STP site during the design basis flood event is exceeded as discussed in FSER section 2.4S.4; the applicant has already requested a departure from the maximum flood level DCD requirement. The staff's review for subsurface hydrostatic loading is divided into two review topics: (1) the maximum groundwater level under normal conditions and all extreme events excluding the "maximum flood level," and (2) the maximum groundwater level during the event resulting in the "maximum flood level." The structural, flood protection, and PRA analyses take into account the DBF. For example, the structural analysis considers saturated conditions from 28 ft-MSL to 40 ft-MSL when evaluating the hydrostatic loads.

DSEA management concludes that all necessary departures have been requested and that the staff's groundwater analysis and corresponding structural, PRA, and flood protection analyses appropriately consider ground water conditions for all conditions including the DBF.

FSER Changes

No changes were made to the FSER.

References

Ahn, Hosung. June 2011. South Texas Project Combined License Application Review: SER with no Open Item Chapter 2.4. Enclosure 1: Re-Analysis of MCR Breach Flood.
Enclosure 2: PNNL Calpack – commented and corrected version by Hosung Ahn.

Baecher, Gregory. November 2011. Independent Technical Review of the Main Cooling Reservoir Dam Breach. University of Maryland.

Irish, Jennifer L. November 2011. Independent Technical Review: South Texas Plant Units 3 and 4 Storm Surge Analysis.

Luettich, Rick. November 2011. External Review of Non-Concurring Issues on the Safety Evaluation Report for the South Texas Project Combined License Application.

MacDonald, T.C., and J. Langridge-Monopolis. May 1984. Journal of Hydraulic Engineering, Breaching Characteristics of Dam Failures. Volume 110, No. 5.

NOAA Technical Report NWS 48. April 1992. SLOSH: Sea. Lake, and Overland Surges from Hurricanes.

NOAA Technical Report NWS 23. September 1979. Meteorological Criteria for Standard Project Hurricane and Probable Maximum Hurricane Windfields. Gulf and East Coasts of the United States. U.S. Department of Commerce, Washington, D.C.

Patev, Robert C. November 2011. Independent Technical Review for the Main Cooling Reservoir Embarkment at the South Texas Project Units 3 & 4. U.S. Army Corps of Engineers.

Resio, Donald T. November 2011. Independent Review of South Texas Plant (STP) Units 3 and 4 Storm Surge Analysis.

State of Colorado, Department of Natural Resources, Division of Water Safety. February 2010. Guidelines for Dam Breach Analysis. Denver, Colo.

Wahl, Tony L. May 2004. Uncertainty of Predictions of Embankment Dam Breach Parameters. Journal of Hydraulic Engineering, Volume 130, No. 5, DOI 10.1061/ (ASCE) 0733-9429(2004)130.5(389).

Wahl, Tony L. July 1998. Predicting Embankment Dam Breach Parameters – A Literature Review and A Needs Assessment. DSO-98-004. Dam Safety Office, U.S. Department of the Interior.

Wahl, T.L., B. Feinberg, D. Gillette, and R.F. Einhellig. November 2011. Review of Main Cooling Reservoir Embankment Breach Analysis – Expansion of South Texas Project Electric Generating Station. U.S. Department of the Interior. Denver, Colo.

U.S. Nuclear Regulatory Commission. August 2012. Section 2.4S Final Safety Evaluation Report for the South Texas Electric Generating Station Units 3 and 4.

Xu, Y. and L.M. Zhang. December 2009. Breaching Parameters for Earth and Rockfill Dams. Journal of Geotechnical and Geoenvironmental Engineering. Volume 135, Number 12. Pages 1957-1970. DOI 10.1061/(ASCE)GT.1943-5606.0000162.

Web Page

http://www.engineeringtoolbox.com/mannings-roughness-d_799.html

Parameter	Applicant	Staff	Dr. Ahn
Initial water Surface elevation in Main Cooling Reservoir (ft MSL)	50.9 (combined effect of normal maximum operating main cooling reservoir water surface elevation, one-half PMP, and 2-year wind waves)	50.9 (the staff agreed with applicant's determination)	50.9
Bottom Elevation of Breach (ft MSL)	29 (elevation of main cooling reservoir service road at base of embankment)	29 (the staff agreed with applicant's determination)	29
Failure Mechanism	Piping (freeboard analysis suggests that overtopping scenario is unlikely)	Piping (staff agreed with applicant's determination; see SER 2.4S.8.4.2)	Piping. Although it should be noted that Enclosure 2 Dr. Ahn's markup of the PNNL calculation package he states that PNNL should change the calculation package to stated that the analysis considers piping or landslide failure.
Storage Volume of the Main Cooling Reservoir (ac-ft)	152,700	152,700	152,700
Depth of water in the Main Cooling Reservoir prior to breach (ft) (h_w)	21.9 (difference of initial water surface elevation and bottom level of breach)	21.9 (the staff verified applicant's determination)	21.9
Initial elevation of formation of pipe (ft MSL)	34	32 (the staff determined this elevation using sensitivity analysis)	34 (assumed based on statement that NWS BREACH parameters with the exception of Manning's n are the same as the applicant's)
Manning's n for embankment material	**0.05**	**0.075 (on upper end of realistic range for conservatism)**	**0.075 (Dr. Ahn estimated the value considering breach conditions, sandy chimney and cement blocks is 0.0775. Manning n value was calculated using the method presented by Chow, which includes accounting for many channel factors including channel size, variation, obstructions, vegetation, and meandering)**
Cohesive strength of embankment material (lb/ft^2)	300	200 (smaller value than that used by the applicant for conservatism)	300

Enclosure

Internal friction angle of embankment material (degrees)	20 (refers to report done by Bechtel Energy Corporation (1984))	15 (the staff determined this value using sensitivity analysis)	20(assumed based on statement that NWS BREACH parameters with the exception of Manning's n are the same as the applicant's)
Embankment length (ft)	**1000**	**4000 (the staff used this value to preclude any artificial constraints on breach formation)**	**3000**
Tailwater cross-section geometry and associated Manning's n	**The tailwater cross-sections used in FLDWAV progressively get wider downstream of the breach. At the breach the cross-section widens from 380 ft to 700 ft from top to bottom and progressive get wider downstream. The tailwater cross-section Manning's n varies from 0.04 at the most upstream cross-section to 0.06 downstream.**	**The staff's sensitivity analysis did not suggest that tailwater cross-section was a dominant factor in development of conservative estimates for breach parameters. For the three cases where the asymptotic limit was reached, the breach width did not attain the full tailwater cross-sectional width initially specified in NWS BREACH model inputs. The staff used a tailwater cross-section Manning's n of 0.06.**	**Tailwater cross-section rectangle shape 3000ft base and 3000ft at the top. Cross-section Manning's n of 0.045 typical; A sensitivity was done between 0.025-0.08 and Dr. Ahn concluded that the DBF was not sensitive to a change in the tailwater cross-section Manning's n as the DBF only changed by 0.36 ft)**
Embankment breach peak discharge (cfs)	**130,000 (estimated using FLDWAV based on breach parameters estimated using Froehlich and MacDonald and Langridge-Monopolis methods)**	**127,929 (estimated using NWS BREACH)**	**336,000-430,000 (Based on three scenarios : 1. FLDWAV using MLM breach width; 2. STP 4745 ft breach width estimate in Rev 0; 3. Empirical Equations that result in the 5 highest Peak outflows**
Embankment breach time-to-peak (hr)	1.7 (estimated using MacDonald and Langridge-Monopolis (1984)).	1.99 (estimated using NWS BREACH)	1.3-1.55 (appears to be based on FLDWAV using breach width for the 3 scenarios described above)
Embankment Breach width (top/bottom) ft	**417 (average) and 380 (bottom) (both estimated using Froehlich (1995))**	**574.3 (top) and 463.6 (bottom) (both estimated using NWS BREACH)**	**1736-4745 (average)**

Maximum flood water surface elevation based on breach discharge within the power block areas (ft MSL)	38.8 (estimated using RMA2 with FLDWAV discharge hydrograph as input based on breach width and time to peak based empirical equations.)	39.04 (estimated using RMA2 with NWS BREACH discharge hydrograph as input)	45.12-46.77ft (based on using FLDWAV and multiple linear regression equation based on FLO-2D analysis of 18 scenarios varying 3 breach widths and 6 time to peak values)
Scour hole	**The applicant estimated that a scour hole with dimensions of 20 ft depth, 203 ft length, and 380 ft width would form downstream of the toe of the Main Cooling Reservoir.**	**The foundation of the Main Cooling Reservoir embankment consists of compacted clays that are resistant to erosion and therefore the staff concluded that formation of a scour hole immediately below the Main Cooling Reservoir embankment is unlikely. The staff also agreed with the applicant that formation of a scour hole below the toe of the Main Cooling Reservoir in an area where native, uncompacted soils are present is likely.**	**Considered three possible cases** **Case 1:No scour** **Case 2: Scour hole forms below Main Cooling Reservoir w/Avg Depth of 10 ft (0 ft at the invert increasing linearly to 20ft at the toe)** **Case 3: Scour hole forms below Main Cooling Reservoir w/Avg Depth of 20 ft (uniform hole depth from invert to the toe)** **In all cases scour would extend from the toe through the distance assumed by STP downstream.**
Sediment available for deposition (ft^3)	The applicant estimates that the Main Cooling Reservoir embankment will contribute approximately 1.9 million ft^3 of material to the flood. The applicant also estimates that the flood following the Main Cooling Reservoir embankment breach will produce a scour hole approximately 20 ft deep, 203 ft long, and 380 ft wide and will therefore contribute approximately 1.5 million ft^3 of material to the flood flow.	The staff estimated the volume of the embankment eroded during the breach to be about 3.1 million ft^3 by doubling the applicant's estimate of the scour hole volume of 1.5 million ft^3 to conservatively account for uncertainty in the dimensions of the postulated scour hole. Therefore, the staff's estimate of total volume of mobilized sediment was 6.2 million ft^3.	Uncertain-Based on discussion in 6.6.1 of Enclosure 1 "the Reanalysis of the Main Cooling Reservoir Breach Flood," it appears sedimentation was not considered. "FLO-2D model has options to simulate various types of flows, including overland flow, channel flow, flood plain, levee, dam and levee breach, hydraulic structures, street flow, rain-run off-infiltration process, sediment transport, mud and debris flow and others. However, the Main Cooling Reservoir FLO-2D model utilizes only the overland flow option and limited options for levee and levee breach because it uses the breach outflow hydrograph simulated by FLDWAV model as upstream boundary inflow."

Maximum flood water surface elevation based on breach discharge within the power block areas accounting for potential deposition of material available from the breach and the scour hole (ft MSL)	**39.2 (estimated using FLDWAV, RMA2, and a bounding sedimentation calculation)**	**39.9 (estimated using NWS BREACH, RMA2, and bounding sedimentation calculation)**	**45.12-46.77ft (based on using FLDWAV and multiple linear regression equation based on FLO-2D analysis of 18 scenarios varying 3 breach widths and 6 time to peak values.)**
Design-basis flood elevation (ft MSL)	**40 (applicant rounded the maximum flood water surface elevation upward)**	**40 (the staff's confirmatory analysis yielded a maximum flood water surface elevation of 39.9 ft MSL, which did not exceed the applicant's chosen design-basis flood elevation (40 ft MSL); therefore, the staff agreed with the applicant's design-basis flood elevation)**	**45 conservative or 47 very conservative**

South Texas Project
Phase 4: SER with no Open Items
Chapter 2.4
Concurrence Sheet

This document will serve as the concurrence page for this chapter and will be placed in the ADAMS package for the chapter. Please initial and date adjacent to your printed name, which will indicate your concurrence.

Participants	Attendance (print name)	Concurrence (initials and date)
Lead Project Manger NRO/DNRL/NGE	George Wunder	05/20/11
Chapter Project Manger NRO/DNRL/NGE	Tekia Govan	5/10/11
Branch Chief NRO/DNRL/NGE	Mark Tonacci	5/12/11
Branch Chief NRO/DSER/RHEB	Richard Raione	for 5/10/11
Licensing Assistant NRO/DNRL/NGE	Bernadette Abeywickrama	BHA 5/16/11
OGC	Laura Goldin and Sara Kirkwood	Via email 5/25/11

OFFICIAL RECORD COPY

Govan, Tekia

From: Goldin, Laura
Sent: Wednesday, May 25, 2011 10:27 AM
To: Govan, Tekia; Ghosh, Anita; Kirkwood, Sara
Cc: Wunder, George; Tonacci, Mark
Subject: RE: STP 2.4 OGC comments - addressed

Tekia,

I reviewed the changes made to incorporate my comments and have NLO.

Thanks,
Laura

From: Govan, Tekia
Sent: Monday, May 23, 2011 4:17 PM
To: Goldin, Laura; Ghosh, Anita; Kirkwood, Sara
Cc: OGCMailCenter Resource; Wunder, George; Tonacci, Mark
Subject: STP 2.4 OGC comments - addressed

Please find attached the staff revisions and comments to address the OGC items raised during your review of the STP SER Section 2.4. Please review. If a call/meeting is necessary please let me know. Otherwise, please let me know when I have OGC's NLO.

The attached titled: T_Govan has sections 2.4.1 – 2.4.7 comments addressed and Ghosh has sections 2.4.8-2.4.14 comments addressed.

Thanks
Tekia

Tekia V. Govan, Project Manager
U.S. Nuclear Regulatory Commission
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Washington DC 20555-0001
301-415-6197
Tekia.Govan@nrc.gov

From: Raione, Richard
Sent: Monday, May 23, 2011 3:35 PM
To: Govan, Tekia
Cc: Ahn, Hosung; Jones, Henry; Tiruneh, Nebiyu; See, Kenneth
Subject:

OGC comments addressed. See attached.

Richard Raione, PG, CPG, CGWP
US NRC, Office of New Reactors
Chief, Hydrologic Engineering Branch
301-415-7190

Govan, Tekia

From: Kirkwood, Sara
Sent: Wednesday, May 25, 2011 11:01 AM
To: Govan, Tekia
Subject: RE: STP 2.4 OGC comments - addressed

Tekia-
I'll go ahead and give you the NLO on the second 1/2 , but comment SBK47 in 2.4s.8.6 still needs to be addressed. I think a confirmatory item has been lost- would you please make sure that it is being mentioned appropriately in the conclusion and tracked (or if it is closed out- that should be reflected in the text.)
Thanks,
Sara

From: Govan, Tekia
Sent: Wednesday, May 25, 2011 10:34 AM
To: Kirkwood, Sara
Cc: Wunder, George; Tonacci, Mark; Ghosh, Anita
Subject: RE: STP 2.4 OGC comments - addressed

Sara:

Could you please let me know the status of your review of the comments addressed for sections 2.4.8-2.4.14?

Thanks
Tekia

Tekia V. Govan, Project Manager
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From: Goldin, Laura
Sent: Wednesday, May 25, 2011 10:27 AM
To: Govan, Tekia; Ghosh, Anita; Kirkwood, Sara
Cc: Wunder, George; Tonacci, Mark
Subject: RE: STP 2.4 OGC comments - addressed

Tekia,

I reviewed the changes made to incorporate my comments and have NLO.

Thanks,
Laura

From: Govan, Tekia
Sent: Monday, May 23, 2011 4:17 PM
To: Goldin, Laura; Ghosh, Anita; Kirkwood, Sara
Cc: OGCMailCenter Resource; Wunder, George; Tonacci, Mark
Subject: STP 2.4 OGC comments - addressed

CONTENTS

ACRONYMS

2DH	2D-horizontal
ABWR	Advanced Boiling-Water Reactor
ac	acre(s)
ac-ft	acre-feet
ADCIRC	Advanced Circulation
ANS	American Nuclear Society
ANSI	American national Standards Institute
BGS	below ground surface
BTP	Branch Technical Position
CFR	Code of Federal Regulations
CFRW	Crane Foundation Retaining Wall
cfs	cubic (foot)feet per second
COL	combined license
COLA	combined license application
COMCOT	Cornell Multigrid Coupled Tsunami Model
CPGCD	Coastal Plains Groundwater Conservation District
CWS	circulating-water system
d	day(s)
DCD	design control document
DBF	design-basis flood
ECL	effluent concentration limit
ECP	essential cooling pond
EOP	emergency operating procedure
ER	Environmental Report
FEMA	Federal Emergency Management Agency
FERC	Federal Energy Regulatory Commission
FM	Farm-to-Market
FSAR	Final Safety Analysis Report
ft	(foot)feet
gal	gallon(s)
g/cc	grams per cubic centimeter
GHB	general head boundary (condition)
gpd/ft^3	gallon(s) per day per cubic foot
gpm	gallon(s) per minute
HD	horizontal dimension
HEC-HMS	Hydrologic Engineering Center Hydrologic Modeling System

HEC-RAS	Hydrologic Engineering Center-River Analysis System
H:V	horizontal versus vertical
HMR	Hydrometeorological Report
hr	hour(s)
in.	inch(es)
kg	kilogram(s)
kg/m^3	kilogram(s) per cubic meter
km	kilometer
km^2	square kilometer(s)
L	liter(s)
Lb	pound(s)
lb/ft^3	pound(s) per cubic foot
LCW	Low Conductivity Waste
LCWPR	Lower Colorado Water Planning Region
LIDAR	Light Detection And Ranging
Lpm	liter(s) per minute
Lps	liter(s) per second
LRS	Little Robbins Slough
LWMS	Liquid Waste Management System
m	meter(s)
M	million
mb	millibar(s)
MDC	Main Drainage Channel
Mi	mile(s)
MLW	mean low water
MOM	Maximum of Maximum
MOST	Method of Splitting a Tsunami
Mph	mile(s) per hour
MSL	(above) mean sea level
NAVD88	North American Vertical Datum of 1988
NCDC	National Climatic Data Center
NGDC	National Geodetic Data Center
NGVD29	National Geodetic Vertical Datum of 1929
nm	nautical mile(s)
NOAA	National Oceanic and Atmospheric Administration
NOS-CO-OPS	(NOAA's) National Ocean Service Center for Operational Oceanographic Product Services
NRC	U.S. Nuclear Regulatory Commission
NRCS	Natural Resources Conservation Service
NSW	National Weather Service
PMF	probable maximum flood

PMH	probable maximum hurricane
PMP	probable maximum precipitation
PMSS	probable maximum storm surge
PMT	probable maximum tsunami
PMWS	probable maximum wind storm
RAI	request for additional information
RMPF	reservoir makeup pumping facility
RSW	reactor service water
s	second(s)
SER	Safety Evaluation Report
SLOSH	Sea, Lake, and Overland Surges from Hurricanes
SRP	Standard Review Plan
SSC	structures, systems, and components
STP	South Texas Project
TDS	total dissolved solids
TS	technical specification
TWDB	Texas Water Development Board
UFSAR	Updated Final Safety Analysis Report
UHS	ultimate heat sink
USACE	U.S. Army Corps of Engineers
USBR	United States Bureau of Reclamation
USGS	U.S. Geological Survey
WES	Waterways Experiment Station

2.4S Hydrologic Engineering

To ensure that a nuclear power plant or plants can be designed, constructed, and safely operated on an applicant's proposed site and in accordance with the U.S. Nuclear Regulatory Commission (NRC or Commission) regulations, NRC staff evaluated the hydrologic impacts on the proposed site. These impacts include the potential for flooding due to precipitation, riverine, and coastal effects. In addition, the staff reviewed the impacts on the site from groundwater flow, ice, and low water effects. These hydrological impacts determine the design-basis flood of a new nuclear power plant and whether flood protection will be required. In addition the staff addressed the potential for the release of radiological material into ground and surface water.

The staff prepared Sections 2.4S.1 through 2.4S.14 of this Safety Evaluation Report (SER) in accordance with the review procedures described in NUREG–0800, using information presented in Section 2.4S of the South Texas Project (STP) Units 3 and 4 combined license (COL) Final Safety Analysis Report (FSAR), which references Revision 4 to the Advanced Boiling-Water Reactor (ABWR) design control document (DCD),,applicant responses to staff requests for additional information (RAIs), and available reference materials (e.g., those cited in applicable sections of NUREG–0800).

2.4S.1 Hydrologic Description

2.4S.1.1 Introduction

This section of the FSAR describes the site and all safety-related elevations, structures, and systems from the standpoint of hydrologic considerations and provides a topographic map showing any proposed changes to natural drainage features.

This SER section provides a hydrologic description of the following specific review areas: (1) the interface of the plant with the hydrosphere including descriptions of site location, major hydrological features in the site vicinity, characteristics related to surface water and groundwater, and the proposed water supply to the plant; (2) hydrological causal mechanisms that may require special plant design bases or operating limitations with regard to floods and water-supply requirements; (3) current and likely future surface-water and groundwater uses by the plant and water users in the vicinity of the site that may affect the safety of the plant; (4) available spatial and temporal data relevant for the site review; (5) alternate conceptual models of the hydrology of the site that reasonably bound hydrological conditions at the site; (6) potential effects of seismic and non-seismic data on the postulated design bases and how they relate to the hydrology in the vicinity of the site and the site region; and (7) any additional information requirements prescribed within the "Contents of Application" sections of the applicable Subparts of *Title 10 of the Code of Federal Regulations* (10 CFR) Part 52. These areas are reviewed in Sections 2.4S.2 through 2.4S.14.

2.4S.1.2 Summary of Application

In Section 2.4S.1 of the FSAR the applicant describes the site and all safety-related elevations, structures, and systems from the standpoint of hydrologic considerations and provides a topographic map showing any proposed changes to natural drainage features.

In addition, in this section, the applicant provides site-specific supplemental information to address COL License Information Item 2.13 identified in DCD Tier 2, Revision 4, Section 2.3.

COL License Information Item

- COL License Information Item 2.13 Hydrologic Description

COL License Information Item 2.13 requires COL applicants to provide a detailed description of all major hydrologic features on or in the vicinity of the site and a specific description of the site and all safety-related elevations, structures, exterior accesses, equipment, and systems from the standpoint of hydrologic considerations.

2.4S.1.3 Regulatory Basis

The associated acceptance criteria are described in Section 2.4.1 of NUREG–0800.

The applicable regulatory requirements for identifying the site location and describing the site hydrosphere are as follows:

- 10 CFR Part 100, as it relates to identifying and evaluating hydrologic features of the site.
- 10 CFR 100.20(c), as it relates to requirements to consider physical site characteristics in site evaluations.
- 10 CFR 52.79(a)(1)(iii), as it relates to the hydrologic characteristics of the proposed site with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

The staff also used the regulatory positions of the following regulatory guides for the identified acceptance criteria:

- Regulatory Guide (RG) 1.27, "Ultimate Heat Sink for Nuclear Power Plants"
- RG 1.102, "Flood Protection for Nuclear Power Plants"

2.4S.1.4 Technical Evaluation

The staff reviewed the information in Section 2.4S.1 of the STP Units 3 and 4 COL FSAR. The staff's review confirmed that the information in the application addresses the relevant information related to site hydrologic description. The staff's technical review of this section includes an independent review of the applicant's information in the FSAR and in the responses to the RAIs. This section describes the staff's evaluation of the technical information in FSAR Section 2.4S.1.

COL License Information Item

- COL License Information Item 2.13 Hydrologic Description

The staff reviewed the hydrologic description of the STP site and vicinity. The staff's review of major hydrological features and descriptions of the site and safety-related elevations, structures, exterior accesses, equipment, and systems is summarized below.

2.4S.1.4.1 Site and Facilities

This section describes the location of the proposed site and the major facilities of the proposed plant.

Information Submitted by Applicant

The STP Units 3 and 4 site is on the west bank of the Colorado River, opposite river kilometer 23.5 (river mile [mi] 14.6) from the Gulf Coast. The STP site is approximately 49.4 square kilometers (km^2) (12,200 acres [ac]) in size including the main cooling reservoir, which has a surface area of approximately 28.3 km^2 (7,000 ac) (see Figure 2.4S.1-1 below). The elevation of the site varies from approximately 4.6 meters (m) (15 feet [ft]) above mean sea level (MSL) south of the main cooling reservoir to approximately 10.4 m (34 ft) MSL near the north edge of the site.

The main cooling reservoir is a manmade reservoir enclosed by a 20-km (12.4- mi) long earthen embankment. The main cooling reservoir is used as the heat sink in a closed-loop cooling system for normal operation of STP Units 1 and 2 and will be similarly used for STP Units 3 and 4. The main cooling reservoir is not a safety-related facility because it will hold no safety-related water for STP Units 3 and 4. The reservoir makeup pumping facility (RMPF), an intake system located on the west bank of the Colorado River, supplies makeup water to the main cooling reservoir. Existing STP Units 1 and 2 use a smaller reservoir, the 0.19-km^2 (46-ac) essential cooling pond (ECP), as the ultimate heat sink (UHS). STP Unit 3 and Unit 4 will each have a UHS consisting of an engineered concrete structure water-storage basin with a dedicated reactor service water (RSW) pump house and dedicated mechanical draft cooling towers. Onsite groundwater wells would be the primary source of makeup water to the UHS basin with the main cooling reservoir as a secondary backup source.

The design-basis flood for the STP site results from a postulated instantaneous breach of a north segment of the main cooling reservoir embankment and is described in detail in FSAR Section 2.4S.4. The applicant determines the design-basis flood elevation to be 12.2 m (40 ft) MSL, which is higher than the normal plant site grade of 10.4 m (34 ft) MSL for STP Units 3 and 4. Safety-related structures, systems, and components (SSCs) require flood protection, which is described in FSAR Section 2.4S.10.

In accordance with the requirements in Appendix A of 10 CFR Part 52, the applicant compares the STP Units 3 and 4 hydrologic site characteristics with the respective envelopes of the ABWR standard plant site design parameters specified in Section 5.0, Table 5.0 of the referenced ABWR DCD Tier 1. The envelope of the ABWR standard site design parameter for a maximum flood level is 1 ft below the plant grade. Because the design-basis flood level at the STP site is higher than the corresponding ABWR standard site design parameter, the applicant identifies this issue as a departure, STP DEP T1 5.0-1, from the certified design.

NRC Staff's Technical Evaluation

The staff conducted a hydrology site audit from March 25, 2008 through March 27, 2008. The site audit included a visit to (1) the STP site and a tour of the RMPF and the barge canal on the Colorado River; (2) the main cooling reservoir, including intake and outfall locations; (3) the STP Units 3 and 4 power block location; and (4) the Little Robbins Slough (LRS), where it crosses the west access road. The staff observed the (1) general site layout, (2) location of Units 3 and 4 in relation to the location of the main cooling reservoir, (3) relief well system on the main cooling reservoir embankment, (4) surface drains that channel surface runoff and relief well discharge into the Colorado River, and (5) main drainage ditch that the applicant proposes to relocate before the construction of STP Units 3 and 4.

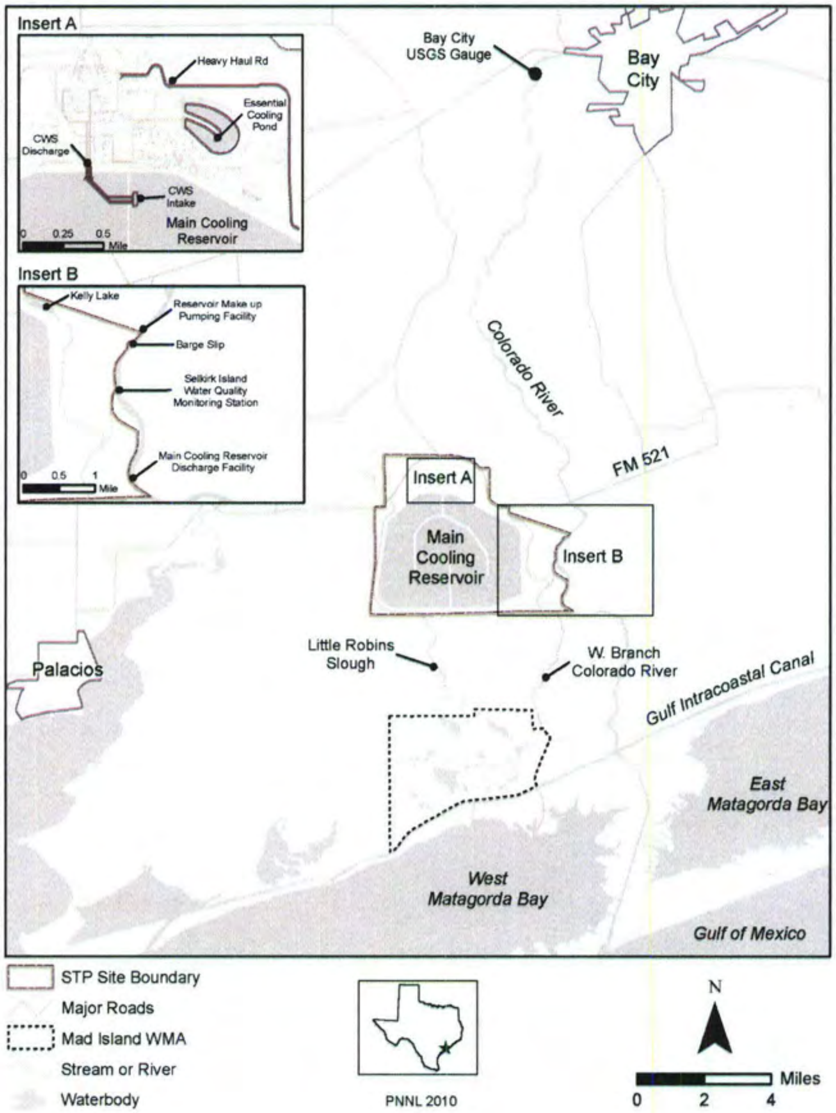

Figure 2.4S.1-1. Map Showing the Location of the STP Site

The staff compared the information from the applicant in FSAR Section 2.4S.1 with publicly available maps and data regarding the STP site and its surrounding region. The STP site is located approximately 14.5 km (9 mi) southwest of Bay City, Texas, and approximately 12.9 km (8 mi) northeast of Palacios, Texas (Figure 2.4S.1-1). The Colorado River flows south on the eastern boundary of the STP site. The West Branch of the Colorado River and the Little Robbins Slough (LRS) are located to the east and west of the main cooling reservoir (Figure 2.4S.1-1). The Matagorda Bay and the Gulf of Mexico are located approximately 19.3 and 24.1 km (12 and 15 mi), respectively, south of the STP Units 3 and 4 location, and the northern tip of Palacios Bay is located about 8 km (5 mi) west of the site.

The staff's evaluation of departure STP DEP T1 5.0-1 is described in SER Sections 2.4S.4 and 2.4S.10.

2.4S.1.4.2 Hydrosphere

This section describes the hydrology in the vicinity of the proposed site, including rivers and streams, lakes and reservoirs, coastal regions, and surface-water and groundwater uses.

Information Submitted by Applicant

The FSAR descriptions of surface water in the vicinity of the STP site include descriptions of the Colorado River Basin, LRS, adjacent drainage basins, shore regions, and surface-water and groundwater uses.

The Colorado River Basin

The Colorado River Basin is 109,603 km^2 (42,318 mi^2) in size, of which 29,534 km^2 (11,403 mi^2) are considered non-tributary. The Upper Colorado River Basin is the portion lying upstream of Lake O.H. Ivie, with an approximate area of 50,857 km^2 (19,636 mi^2). The Lower Colorado River Basin is the remaining portion, 58,746 km^2 (22,682 mi^2) in area, from Lake O.H. Ivie to the Gulf Coast.

The climate of the Colorado River Basin is warm and temperate with dry winters and humid summers. Spring and fall are wet seasons with rainfall peaks in May and September. Convective thunderstorms, typically of short duration and high intensity, dominate spring rainfall. Fall precipitation results from tropical storms and hurricanes that originate in the Caribbean Sea and the Gulf of Mexico. Annual rainfall in the region varies from 112 centimeters (cm) (44 inches [in.]) at the coast to 61 cm (24 in.) inland.

Stream-flow data in the Colorado River Basin have been measured since the early 1900s. There has been a major drought in the basin in almost every decade of the twentieth century. Three major statewide droughts have occurred between 1941 and 1970: from 1947 to 1948, from 1950 to 1957 (the most severe), and from 1960 to 1967.

The Colorado River Basin has 30 dams with individual storage capacities exceeding 12.3 million cubic meters (m^3) (10,000 ac-ft) (FSAR Table 2.4S.1-1). Although the dams in the Colorado River Basin were constructed primarily for flood control, they are also used to supply water. Six of the dams on the Lower Colorado River are operated by the Lower Colorado River

Authority (LCRA). These six dams—Buchanan, Inks, Wirtz, Starcke, Mansfield, and Tom Miller—impound the six Highland Lakes: Buchanan, Inks, Lyndon B. Johnson, Marble Falls, Travis, and Austin, respectively. Of these, the Buchanan and Mansfield dams are the two major structures on the Colorado River that may influence flood conditions near the STP site. Both dams were designed or upgraded to safely pass their respective probable maximum floods. Mansfield Dam is currently the most downstream major control structure on the Colorado River and impounds Lake Travis. With a storage capacity of 3,976 million m^3 (3,223,000 ac-ft), Lake Travis is the largest reservoir in the Colorado River Basin. Mansfield Dam and Lake Travis provide most of the floodwater storage capacity in the basin.

Lakes Travis and Buchanan also supply water for communities, industry, irrigation, and aquatic life with water-supply storage capacities of approximately 1,397 and 1,079 million m^3 (1,132,400 and 875,000 ac-ft), respectively.

Wider and flatter lateral slopes characterize the Colorado River flood plain downstream from the city of Columbus compared to the flood plain upstream of the city. The flood plain downstream of the city is also characterized by no discernible valley, and interbasin spillage occurs during high flood discharges.

Downstream of Mansfield Dam are seven U.S. Geological Survey (USGS) stream-flow gauge stations (FSAR Table 2.4S.1-3 and FSAR Figure 2.4S.1-8). The stream-flow gauge closest to the STP site on the Colorado River is located approximately 25.7 km (16 mi) upstream, 3.7 km (2.3 mi) west of Bay City (Texas) at river km 52.3 (river mile 32.5). Stream-flow records at the Bay City gauge have existed since April 1948.

<u>Little Robbins Slough</u>

LRS is an intermittent stream located about 14.5 km (9 mi) northwest of Matagorda, Texas, with a length of approximately 10.5 km (6.5 mi) before it joins Robbins Slough. Robbins Slough is a brackish marsh south of the STP site that flows approximately 6.4 km (4 mi) to the Gulf Intracoastal Waterway. During construction of the main cooling reservoir, LRS was relocated to a channel west of the main cooling reservoir. The relocated LRS flows parallel to the west embankment of the main cooling reservoir and joins its natural course approximately 1 mi east of the southwest corner of the main cooling reservoir,

<u>Adjacent Drainage Basins</u>

The Colorado-Lavaca River Basin is located west of the Colorado River Basin in the coastal region and includes the Tres Palacios Creek, which is not a tributary to the Colorado River or to the Lavaca River. The Colorado-Lavaca River Basin drains into the Tres Palacios Bay. During high flood discharges, such as during the 1931 flood, the floodwaters from the Colorado River overflow the eastern basin ridge into Caney Creek near Wharton, Texas, which is in the San Bernard River Basin. Floodwaters from the Colorado River Basin occasionally spill west into the Colorado-Lavaca Basin.

<u>Shore Regions</u>

The STP site is located approximately 16.9 km (10.5 mi) from Matagorda Bay, approximately 27.2 km (16.9 mi) from the Gulf of Mexico, and approximately 120.7 km (75 mi) from the continental shelf. The Matagorda Peninsula shoreline retreats landward or advances seaward in response to various hydrologic, meteorologic, and climatic factors combined with engineering activities.

The Matagorda Peninsula is a classic microtidal, wave-dominated coastline. The mean diurnal tide varies by approximately 0.6 m (2.1 ft). Based on 20 years of observations, a University of Texas study (Gibeaut et al., 2000) estimated the mean significant wave height of 1 m (3.3 ft) with a mean peak wave period of 5.7 seconds at a location 40 km (24.9 mi) east of the Colorado River entrance in a water depth of 25.9 m (85 ft). Gibeaut et al., (2000) also estimated that the shoreline segment of the Matagorda Peninsula 2.6 km (1.6 mi) southwest of the Colorado River is retreating at a rate of 0.5 to 2.0 m (1.6 to 6.4 ft) per year, whereas the shoreline from this point to the mouth of the river displays a long-term advance. The shoreline northeast of the mouth of the Colorado River only shows a slight long-term advance.

The Colorado River discharged directly into the Gulf through a channel dredged across the Matagorda Peninsula in 1936 after the 1929 removal of a log jam in the Colorado River. In the early 1990s, the U.S. Army Corps of Engineers (USACE) constructed jetties on each side of the river's entrance and dredged an entrance channel. In 1993, the USACE constructed a diversion channel to discharge the Colorado River into the Matagorda Bay. The former river channel is now a navigation channel that connects the IntraCoastal Waterway to the Gulf.

Tropical storms and hurricanes are very common in this region. From 1900 to 2005, 33 major hurricanes of Category 3 and above made landfall on the Texas coast. The applicant states that the expected frequency of occurrence of major hurricanes is approximately once every 3 years.

Surface-Water Use

The Lower Colorado Water Planning Region (LCWPR), or Region K, comprises a total of 15 counties in Texas including Matagorda County, the location of the STP site. Ten aquifer systems and six river and coastal basins form the sources of the water supply to Region K, with the Colorado River representing the largest source of surface water. The Lower Colorado Regional Water Planning Group (2006) estimated the total annual water supply in Region K to be 1,604 million m^3 (1.3 million ac-ft), with a 73-percent contribution from surface-water sources.

The Texas Commission on Environmental Quality maintains a Water Rights Database that contains details of all active and inactive surface-water rights permits and contracts. The LCWPR designates the LCRA and the city of Austin as "wholesale water providers," because they provide a significant amount of water for municipal and manufacturing uses within the region. FSAR Table 2.4S.1-4 lists active surface-water users in Matagorda County. There are no known surface-water users downstream of the STP site.

Groundwater

FSAR Section 2.4S.12 describes local and regional groundwater characteristics, groundwater users, groundwater well locations, and withdrawal rates. Section 2.4S.12.2 of this report summarizes the applicant-provided groundwater-related information.

NRC Staff's Technical Evaluation

The staff reviewed the applicant's information in FSAR Section 2.4S.1. The staff's independent review and determinations regarding the hydrosphere are described below.

The applicant describes the plant's water demands in Environmental Report (ER) Section 3.3. The UHS system provides water for the safety-related cooling of STP Units 3 and 4. Onsite wells primarily provide makeup water to the engineered UHS basins. During the limited peak demand described in ER Section 3.2, the main cooling reservoir provides supplementary water to the UHS basin, as needed. Surface-water and groundwater sources are not safety-related because the engineered UHS basins of each unit have a sufficient capacity to provide a 30-day cooling-water supply to the UHS without the need for any makeup or blowdown.

It is important to note that the FSAR hydrology sections mostly rely on the National Geodetic Vertical Datum of 1929 (NGVD29) as the referenced vertical datum, and the term MSL is based on the NGVD29. In a few exceptional cases, the applicant uses data referenced in the North American Vertical Datum of 1988 (NAVD88) when referring to a few studies conducted by others. There is a small difference of 0.05 m (0.16 ft) between NGVD29 and NAVD88 near the STP site.

The staff reviewed the applicant's description of the hydrosphere in the vicinity of the site and determined that the description is satisfactory. The staff used the NGVD29-based MSL to reference elevations in this report.

The Colorado River Basin

The Colorado River Basin (Figure 2.4S.1-2) is approximately 109,603 km^2 (42,318 mi^2) in size (LCRWPG 2006). The Lower Colorado River Basin is the portion downstream of Lake O.H. Ivie. Approximately 90 percent of the contributing area of the basin lies upstream of the Mansfield Dam near Austin, Texas (LCRWPG 2006). The STP site is located on the west bank of the Colorado River at river kilometer 23.5 (river mile 14.6).

The discharge of the Colorado River near the site is measured at USGS gauge 08162500, near Bay City, Texas. Available stream-flow discharge data at this gauge have been gathered since May 1, 1948.

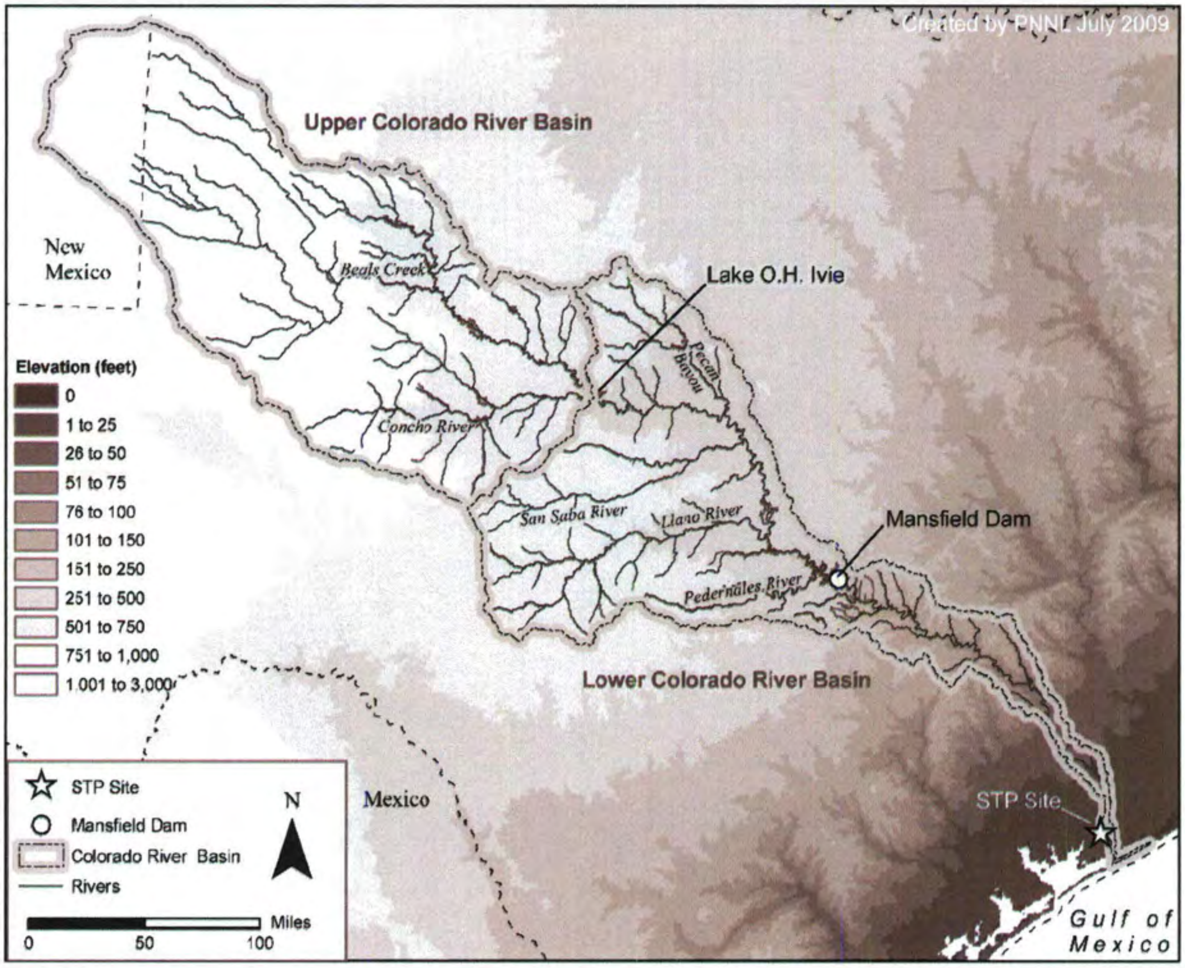

Figure 2.4S.1-2. The Colorado River Basin. (The point of demarcation between the upper and the lower basin is Lake O.H. Ivie. Mansfield Dam and the STP site are also shown.)

The stream flow in the Colorado River downstream of Austin, Texas, is regulated by releases from the Mansfield Dam. The LCRA operates six dams (Buchanan, Inks, Wirtz, Starcke, Mansfield, Tom Miller [LCRA, 2009]) and six respective highland lakes (Buchanan, Inks, Lyndon B. Johnson, Marble Falls, Travis, Austin). Lake Buchanan has a storage capacity of 1,078 million m^3 (875,566 ac-ft) and is used to supply water and to generate hydroelectric power. Lake Travis has a storage capacity of 1,396 million m^3 (1,131,650 ac-ft) and is used to supply water, manage floods, and generate hydroelectric power. The combined water-storage capacity of the six highland lakes is 2,695 million m^3 (2,184,777 ac-ft) (LCRA, 2009). Mansfield Dam provides the most downstream flood-control reservoir in the Colorado River Basin. The broad floodplain in the Lower Colorado Basin has a relatively flat gradient. Interbasin spillage between the Lower Colorado Basin and its adjacent basins can occur during floods because of a lack of steep ridges that separate the subbasins.

Little Robbins Slough

LRS is an intermittent stream that originates approximately 3.2 km (2 mi) northwest of the STP site and has a drainage area of approximately 10.4 km^2 (4 mi^2). During the construction of existing Units 1 and 2 and the main cooling reservoir, the original course of the slough was relocated to the west of the main cooling reservoir. The relocated channel runs along the western edge of the main cooling reservoir embankment, turns east at the southwest corner of the main cooling reservoir embankment, and rejoins its natural course approximately 1.6 km (1 mi) east of the southwest corner of the main cooling reservoir embankment. The LRS flows into Robbins Slough, which is a brackish marsh that joins the Gulf Intracoastal Waterway approximately 6.4 km (4 mi) to the south (Figure 2.4S.1-1). There is no known stream-flow monitoring of the slough.

Adjacent Drainage Basins

The Lower Colorado River Basin is flanked by the Colorado-Lavaca River Basin to the west and the San Bernard Coastal Basin to the east. Flat, wide floodplains and a lack of well-defined basin ridges characterize the terrain near the STP site.

Shore Regions

The Matagorda Peninsula separates the Matagorda Bay from the Gulf, but the southwest portion of the bay is open to the Gulf of Mexico. The shoreline of Matagorda Bay along the Gulf Coast has changed constantly as the result of a combination of hydrologic and meteorological processes, in addition to engineering activities. The shore region of the Matagorda Bay is also affected by waves generated by tropical storms and hurricanes. The hydrologic features of the shore region have also been altered by a series of engineering modifications. After the removal of a log jam on the Colorado River in 1929, the Colorado River directly discharged into the Gulf through a channel dredged across the peninsula in 1936. Beginning in the 1990s, the USACE constructed jetties on each side of the river entrance and dredged an entrance channel. In 1993, the USACE constructed a diversion channel that directs the flow of the Colorado River into the West Matagorda Bay. The former river channel is now a navigation channel connected to the Gulf Intracoastal Waterway. The Gulf Intracoastal Waterway is a 2090-km (1300-mi) long manmade canal that runs along the Gulf of Mexico from Brownsville, Texas, to St. Marks, Florida (Texas Department of Transportation 2007).

Surface-Water Use

Water is withdrawn from the Colorado River to support the operations of existing Units 1 and 2 at the STP site. The withdrawn water is used to replace water lost from the main cooling reservoir due to natural and forced evaporation, seepage, and occasional discharge to maintain water quality for the circulating-water systems (CWSs). The main cooling reservoir and the water withdrawal system from the Colorado River will continue to operate as a similar system to support the operations of STP Units 3 and 4. However, water withdrawal from the Colorado River is not a safety-related activity or essential to plant operation, because the engineered UHS has sufficient capacity to operate the plant for 30 days without supplementing its water storage.

The Main Cooling Reservoir

The predominant surface-water feature near the STP site is the main cooling reservoir, a manmade lake impounded by earthen embankments that was constructed on the natural ground surface immediately south of the existing facility. The main cooling reservoir is part of the closed-loop cooling system for STP Units 1 and 2 and acts as the normal heat sink for waste heat generated during the operations of these units. The main cooling reservoir is currently operated to dissipate waste heat from the operations of existing Units 1 and 2, primarily via evaporation, which results in some water loss from the main cooling reservoir. To support the operations of STP Units 1 and 2, the normal maximum water surface elevation of the main cooling reservoir is 14.3 m (47 ft) MSL.

In addition to evaporation, water is lost from the main cooling reservoir due to seepage. About 770 relief wells were installed along the main cooling reservoir embankment during the construction of the embankment to relieve the hydrostatic pressure caused by levee seepage. These relief wells intercept and divert a portion of the groundwater seepage away from the main cooling reservoir. Water loss from the main cooling reservoir results in a buildup of total dissolved solids within the reservoir. The RMPF, located on the west bank of the Colorado River, withdraws makeup water from the river. The main cooling reservoir has a seven-port discharge facility that operates using variable discharge rates ranging from 2.3 to 8.7 cubic meters per second (m^3/s) (80 to 308 cubic feet per second [cfs]) (see ER Section 3.4.2.2). Each port is equipped with a gated valve. A buried pipe, approximately 1.8 km (1.1 mi) in length, conveys water from the reservoir to the discharge ports installed in the Colorado River. The main cooling reservoir also has a spillway near its southeast corner that allows the release of excess water from the main cooling reservoir into the Colorado River during heavy precipitation events. The spillway contains gates that can be manually opened to release water to the Colorado River through a 1,591-m (5,220-ft) -long channel. According to the FSAR of Units 1 and 2 (Version 13, Subsection 2.4.8.2), the spillway capacity is about 86.4 m^3/s (3,050 cfs) at the main cooling reservoir water level of 15.2 m (50 ft) MSL. Both the discharge and spillway facilities are non-safety-related structures, and the addition of STP Units 3 and 4 will have no effect on the operating rules or design of the facilities.

The main cooling reservoir will be part of the closed-loop cooling system of STP Units 3 and 4 during normal operations. To support the operation of STP Units 3 and 4, the applicant will raise the main cooling reservoir normal maximum water surface elevation to 14.9 m (49 ft) MSL.

Groundwater

Section 2.4S.12 of this report describes the staff's review of groundwater characteristics, groundwater users, groundwater well locations, and withdrawal rates.

2.4S.1.5 Post Combined License Activities

There are no post-COL activities related to this section.

2.4S.1.6 Conclusion

The staff performed an independent review of the applicant's information in FSAR Section 2.4S.1.. The applicant presents and substantiates information relative to the hydrologic description in the vicinity of the site and site regions important to the design and siting of this plant. The staff's review found that the applicant has considered the appropriate site phenomena for establishing the design bases for SSCs important to safety and no outstanding information is required to be addressed in this section. The staff accepted the applicant's approaches used to describe the hydrologic phenomena in the vicinity of the site and site regions.

Accordingly, the staff concluded that the identification and consideration of the safety-related hydrology in the vicinity of the site and site regions are acceptable and meet the requirements of 10 CFR 52.79 and 10 CFR 100.20(c). The information addressing the COL Information Item 2.13 is acceptable.

2.4S.1.7 References

Gibeaut, J.C., White, W.A., Hepner, T., Gutierrez, R., Tremblay, T.A., Smyth, R., and Andrews, J., "Texas Shoreline Change Project – Gulf of Mexico Shoreline Change from the Brazos River to Pass Cavallo, Bureau of Economic Geology," The University of Texas, Austin, 2000.

Lower Colorado Regional Water Planning Group, "*2006 Region 'K' Water Plan for the Lower Colorado Regional Water Planning Group*," January 2006; http://www.twdb.state.tx.us/rwpg/2006_RWP/RegionK/.

Lower Colorado River Authority, "*LCRA Dams Form the Highland Lakes,*" 2009; http://www.lcra.org/water/dams/index.html ; accessed March 30, 2009.

South Texas Nuclear Operating Company, "South Texas Project Combined License Application," Final Safety Analysis Report, Revision 0, Part 2, 2007.

Texas Department of Transportation, "Gulf Intracoastal Waterway, Legislative Report to the 80th Texas Legislature," Austin, Texas, 2007.

2.4S.2 Floods

2.4S.2.1 Introduction

This section of the FSAR discusses the historical flooding at the proposed site or in the region of the site, and summarizes and identifies the individual types of flood-producing phenomena and combinations of flood-producing phenomena considered in establishing the flood design bases for safety-related plant features. This section also covers the potential effects of local intense precipitation.

This SER section provides a review of the following specific areas: (1) a description of the flood history, (2) flood design considerations, and (3) the effects of local intense precipitation.

2.4S.2.2 Summary of Application

In Section 2.4S.2, the applicant addresses the information related to site and regional flood causal mechanisms. In addition, in this section, the applicant provides site-specific supplemental information to address COL License Information Item 2.14 identified in DCD Tier 2, Revision 4, Section 2.3.

COL License Information Item

- COL License Information Item 2.14 Floods

COL License Information Item 2.14 requires COL applicants to provide site-specific information related to historical flooding and the potential for flooding at the plant site, including flood history, flood design considerations, and the effects of local intense precipitation.

2.4S.2.3 Regulatory Basis

The associated acceptance criteria, are in Section 2.4.2 of NUREG–0800.

The applicable regulatory requirements for identifying floods are as follows:

- 10 CFR Part 100, as it relates to identifying and evaluating hydrological features of the site. The requirement to consider physical site characteristics in site evaluations is specified in 10 CFR 100.20(c).

- 10 CFR 52.79(a)(1)(iii), as it relates to the hydrologic characteristics of the proposed site with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

In addition, the staff used the regulatory positions of the following regulatory guides for the identified acceptance criteria:

- RG 1.27, "Ultimate Heat Sink for Nuclear Power Plants"

- RG 1.59, "Design Basis Floods for Nuclear Power Plants," as supplemented by best current practices

- RG 1.102, "Flood Protection for Nuclear Power Plants."

2.4S.2.4 Technical Evaluation

The staff reviewed the information in Section 2.4S.2 of the STP Units 3 and 4 COL FSAR. The staff's review confirmed that the information in the application addresses the relevant information related to site floods. The staff's technical review of this application included an independent review of the applicant's information in the FSAR and in the responses to the RAIs. The staff supplemented this information with other publicly available sources of data.

This section describes the staff's evaluation of the technical information in FSAR Section 2.4S.2.

COL License Information Item

- COL License Information Item 2.14 Floods

The staff reviewed site-specific information related to historical flooding and the potential for flooding at the plant site, including flood history, flood design considerations, and the effects of local intense precipitation.

2.4S.2.4.1 Flood History

This section describes the historical floods at and in the vicinity of the proposed site.

Information Submitted by Applicant

Flooding near the STP site from natural events includes flooding in the Colorado River, hurricane-induced storm surges, dam and levee breaches, tsunamis, and local flooding in the LRS.

The applicant states in the FSAR Section 2.4S.2 that there are no records of stream flow or stage for the LRS. Using a local probable maximum precipitation (PMP) event, the applicant estimates the local floods that could potentially pose a hazard to safety-related SSCs of STP Units 3 and 4.

The USGS maintains and operates a network of stream gauges at the Colorado River near the vicinity of the STP site. The three gauges closest to the STP site are at Bay City (USGS gauge number 08162500), Wharton (USGS gauge number 08162000), and Columbus (USGS gauge number 08161000). The Bay City and Wharton gauges are more representative of the stream-flow conditions near the STP site because floodplain characteristics upstream of Columbus are different from those near the STP site. The Bay City and Wharton gauges are located approximately 50 to 80.5 km (16 and 50 mi) upstream of the STP site, respectively.

The applicant presents the annual peak stream-flow data at Bay City (for water years 1940 and 1948 through 2006) and at Wharton (water years 1919 through 2006) in FSAR Tables 2.4S.2-1 and 2.4S.2-2, respectively. Flood discharges at these gauges are affected by regulation from several upstream dams. Lake Travis, which was impounded by the construction of Mansfield Dam in 1942, is the largest impoundment in the Colorado River Basin. The highest observed peak discharges at the Bay City and Wharton gauges since the construction of Mansfield Dam are 2381.4 m^3/s (84,100 cfs) on June 26, 1960, and 74,800 cfs on October 23, 1960[1] respectively. The historical peak discharge at the Wharton gauge before the construction of Mansfield Dam is 4502.4 m^3/s (159,000 cfs) on June 20, 1935. The highest recorded flood elevations at the Bay City gauge are 17.1, 16.9 and 16.8 m (56.1, 55.4, and 55.0 ft) MSL in 1913, 1922, and 1929, respectively, before the construction of Mansfield Dam. After the construction of Mansfield Dam, the highest flood elevations at the Bay City gauge were 14.1 and 11.8 m (46.4 and 38.67 ft) MSL in 1960 and 1995 water years, respectively.[2]

During the study of the Colorado River Flood Damage Evaluation Project of the USACE and the LCRA in early 1990s, Halff Associates, Inc. (1992) estimated a flood elevation of 6.4 m (21.0 ft) MSL corresponding to the 2316.3 m^3/s (81,800 cfs) discharge on October 24, 1998, at the Farm-to-Market (FM) 21 Bridge crossing.

At a recently established USGS stream-flow gauge on the Colorado River Bypass Channel near Matagorda (USGS gauge number 08162506), the maximum recorded water surface elevation in the East Colorado River for the period of October 1999 to May 2007 was 2.1 m (7.05 ft) MSL, with a corresponding stream-flow discharge of 2064.3 m^3/s (72,900 cfs) at the Bay City gauge.

FSAR 2.4S.2 states that there are no reported events of ice sheet formation or ice jams for the Colorado River at the STP site or the LRS.

The estimated flood levels from the postulated breach of the main cooling reservoir are higher than the site grade. As a result, the applicant has identified a departure, STP DEP T1 5.0-1, from the certified design. The applicant provides information to address COL License Information Item 2.14 from the generic DCD.

NRC Staff's Technical Evaluation

The staff reviewed the applicant's data in FSAR Section 2.4S.2 regarding historical flooding. The staff independently obtained annual peak flow data for the Wharton and Bay City USGS stream-flow gauges. The staff plotted the historical peak flow data for the two gauges in Figures 2.4S.2-1 and 2.4S.2-2.

Based on these data, the staff determined that the historical maximum peak discharges at the Wharton and Bay City USGS gauges are 4,502.4 m^3/s (159,000 cfs) on June 30, 1935 and 2,381.4 m^3/s (84,100 cfs) on June 26, 1980, respectively. Mansfield Dam was constructed in 1942. Before 1942, the peak discharges at the Wharton USGS gauge have shown higher values ranging from 356.8 to 4,502.4 m^3/s (12,600 to 159,000 cfs), with a mean of 1,757.5 m^3/s (62,067 cfs). Since 1942, the peak discharges have ranged from 108.2 to 2,118.1 m^3/s (3,820 to 74,800 cfs), with a mean of 829.9 m^3/s (29,309 cfs). These discharge value estimates are based on recorded stages at each gauge station. The stages before 1942 ranged from 3.5 to

[1] FSAR Section 2.4S.2 has a typographical error. The correct date for the peak discharge of 74,800 cfs at the Wharton gauge is October 23, 1998.

[2] There are other water years in which floodwater elevation has exceeded 39 ft above MSL at the Bay City gauge. These are described in the staff's technical evaluation.

15.8 m (11.5 to 51.9 ft) MSL. The stages after 1942 ranged from 1.7 to 14.8 m (5.7 to 48.7 ft) MSL.

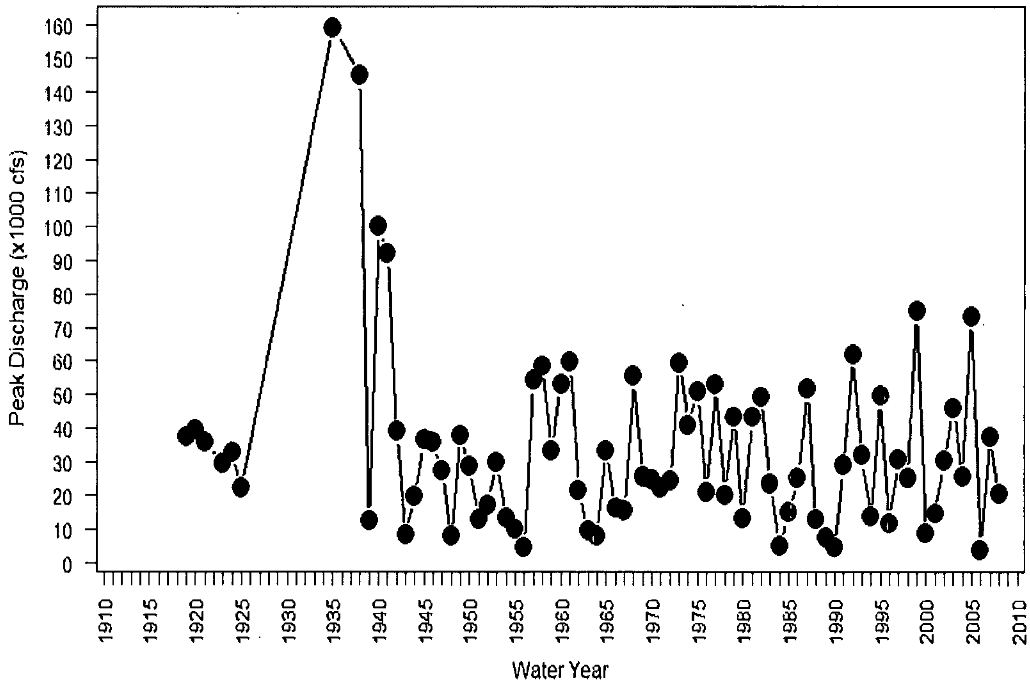

Figure 2.4S.2-1. Peak Stream-Flow Discharge in the Colorado River at the Wharton USGS Gauge

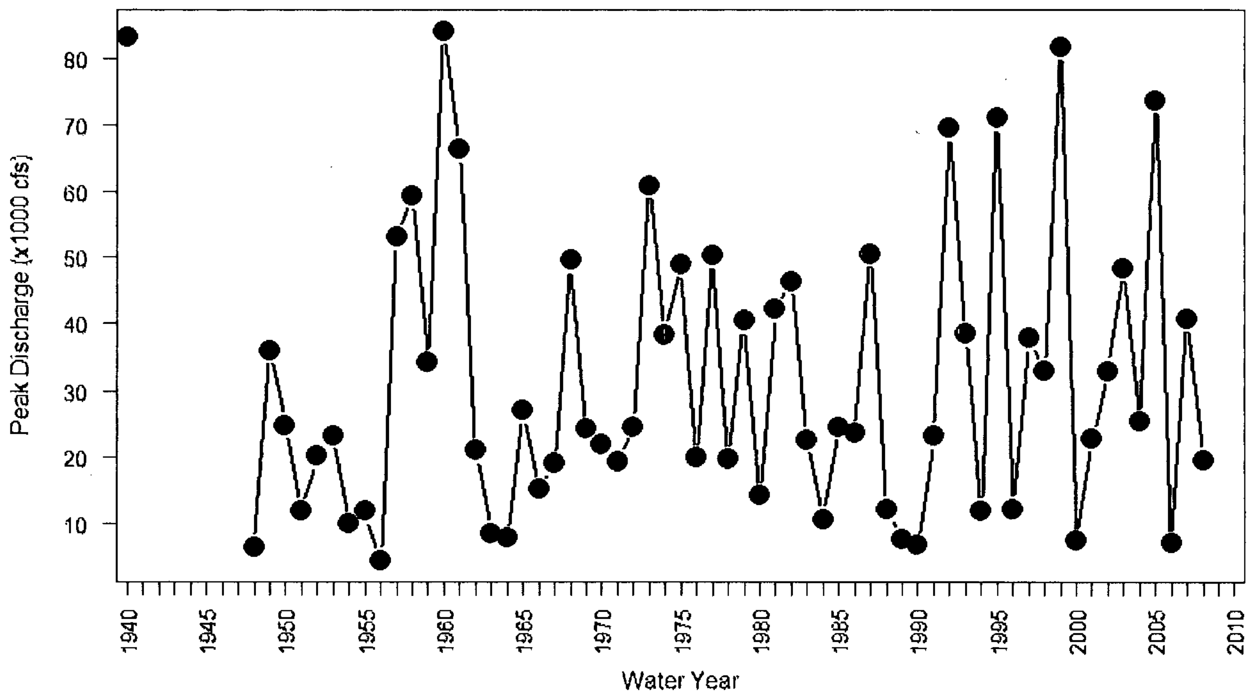

Figure 2.4S.2-2. Peak Stream-Flow Discharge in the Colorado River at the Bay City USGS Gauge

Before 1942, recorded discharges at the Bay City USGS gauge are fully available only in 1940. The annual peak discharge was 2,358.8 m^3/s (83,300 cfs) on July 4, 1940, and the corresponding water level was 14.2 m (46.6 ft) MSL. However, the gauge height during peak stream-flow discharges at the Bay City gauge show consistently higher values ranging from 14.2 to 17.1 m (46.6 to 56.1 ft) MSL. After 1942, the gauge heights during peak stream-flow discharge ranged from 3.8 to 14.1 m (12.5 to 46.4 ft) MSL. Table 2.4S.2-1 shows the maximum gauge heights recorded since 1942.

Table 2.4S.2-1. Ten Highest Water Levels Recorded at the Bay City USGS Gauge Since Construction of the Mansfield Dam in 1942

Date (Water Year)	Peak Discharge (m^3/s) / (cfs)	Water Level (m / ft MSL)
06/26/1960 (1960)	2,381.4 / 84,100	14.1 / 46.4
09/15/1961 (1961)	1,880.2 / 66,400	13.4 / 44.1
10/17/1957 (1958)	1,676.4 / 59,200	13.0 / 42.8
05/01/1957 (1957)	1,500.8 / 53,000	12.7 / 41.8
11/27/2004 (2005)	2,089.8 / 73,800	12.7 / 41.7
10/24/1998 (1999)	2,316.3 / 81,800	12.5 / 41.0
12/27/1991 (1992)	1,970.9 / 69,600	11.9 / 38.9
06/15/1973 (1973)	1,721.7 / 60,800	11.8 / 38.7
10/20/1994 (1995)	2,013.3 / 71,100	11.8 / 38.7
06/26/1968 (1968)	1,401.7 / 49,500	11.4 / 37.5

2.4S.2.4.2 Flood Design Considerations

This section describes the scenarios used to determine the design-basis flood at the STP site.

Information Submitted by Applicant

The applicant determines the design-basis flood elevation at the STP site from several scenarios, including the probable maximum flood (PMF) of streams and rivers, potential dam failures, probable maximum surge and seiche flooding, probable maximum tsunamis, flooding due to ice effects, and the potential for flooding caused by channel diversions. The respective FSAR sections describe these flood scenarios. The applicant considers combinations of appropriate conditions with flood scenarios such as wind-generated waves and tidal levels as recommended by the American National Standards Institute (ANSI)/American Nuclear Society (ANS)-2.8–1992 (ANS, 1992).

The applicant estimates the design-basis floodwater surface elevation at the STP site from the postulated breach of the main cooling reservoir embankment. The design-basis flood elevation of 12.2 m (40 ft) MSL is above the site grade and the ground-floor elevation of safety-related SSCs for STP Units 3 and 4. Therefore, all STP Units 3 and 4 SSCs in the power block area below the elevation of 12.2 m (40 ft) MSL will require appropriate flood-protection measures, such as watertight doors and components that will prevent any floodwater intrusion into safety-related areas of the plant. The UHS and the RSW pump house are located at the UHS tower basin and are watertight below the floor slab elevation at 15.2 m (50 ft) MSL. Therefore, they will not need flood protection. FSAR Section 2.4S.10 discusses flood-protection requirements.

The applicant incorporates by reference Section 2.1 of the certified ABWR DCD referenced in Appendix A of 10 CFR Part 52. Due to flood levels from the postulated breach of the main cooling reservoir at higher than the site grade, the applicant identifies a departure, STP DEP T1 5.0-1, from the certified design. The applicant provides information to address COL License Information Item 2.14 from the generic DCD.

NRC Staff's Technical Evaluation

The staff reviewed the applicant's description of flooding mechanisms in FSAR Section 2.4S.2 and compared them to the applicable guidance in NUREG-0800, Section 2.4.2. The staff determined that the applicant has considered all plausible flooding mechanisms at the STP site. The corresponding section of this SER describes the staff's review of the individual flooding mechanisms and their flooding potential. After reviewing the applicant's submittals and the staff's independent confirmatory analyses in Sections 2.4S.2, 2.4S.3, 2.4S.4, 2.4S.5, 2.4S.6, and 2.4S.10 of this SER, the staff determined that the maximum floodwater surface elevation at the STP Units 3 and 4 site would be caused by a postulated failure of the northern main cooling reservoir embankment. The staff confirmed in Section 2.4S.4 of this SER that the design-basis maximum water surface elevation at the STP Units 3 and 4 site is 12.2 m (40 ft) MSL.

2.4S.2.4.3 Effects of Local Intense Precipitation

This section describes the estimation of local intense precipitation and its effects on the safety-related SSCs of STP Units 3 and 4.

Information Submitted by Applicant

Probable Maximum Precipitation Depths

The applicant estimates the design basis for local intense precipitation, which is the all-season, 2.60-km^2 (1-mi^2) PMP from the U.S. National Weather Service (NWS) Hydrometeorological Reports (HMRs) No. 51 and 52 (Schreiner and Riedel 1978; Hansen et al., 1982). FSAR Table 2.4S.2-4 lists the values of the PMP depths, which are reproduced below in Table 2.4S.2-1.

The 1-hour and 5-minute local PMP depths of 50.3 cm (19.8 in.) and 16.3 cm (6.4 in.), respectively, exceed the corresponding ABWR DCD values of 49.3 and 15.7 cm (19.4 and 6.2 in.), respectively. The applicant identifies this exceedance as a departure, STP DEP T1 5.0-1, from the certified design. Justification for the departure is discussed in FSAR Table 2.0-2. Standard ABWR Seismic Category I structures are designed with roofs without parapets or with parapets and scuppers that supplement roof drainage to minimize the accumulation of precipitation on the roofs. Site-specific Seismic Category I structures, such as RSW pump houses, are designed without parapets to minimize the ponding of water. Therefore, the applicant argues that an exceedance of 1 cm per hour (cm/hr) (0.4 in./hr) in precipitation rate will not result in a substantial increase in roof design load and therefore, will not affect the design of these structures.

Table 2.4S.2-1 Local Intense Precipitation at the STP Site (Adapted from FSAR Table 2.4S.2-4)

PMP Duration and Area	6-hr, 10-mi^2 Ratio	1-hr, Point Ratio	Source	PMP Depth (cm) / (in.)
72 hr, 10 mi^2	–	–	HMR 51 - Fig. 22	141.5 / 55.7
48 hr, 10 mi^2	–	–	HMR 51 - Fig. 21	131.6 / 51.8
24 hr, 10 mi^2	–	–	HMR 51 - Fig. 20	119.6 / 47.1
12 hr, 10 mi^2	–	–	HMR 51 - Fig. 19	96.0 / 37.8
6 hr, 10 mi^2	–	–	HMR 51 - Fig. 18	81.3 / 32.0
3 hr	–	–	Fitted from FSAR Figure 2.4S.2-3	75.4 / 29.7
2 hr	–	–	Fitted from FSAR Figure 2.4S.2-3	67.6 / 26.6
1 hr, point	0.62	–	HMR 52 - Fig. 23	50.3 / 19.8
30 min, point	–	0.73	HMR 52 - Fig. 38	36.8 / 14.5
15 min, point	–	0.50	HMR 52 - Fig. 37	25.1 / 9.9
5 min, point	–	0.32	HMR 52 - Fig. 36	16.3 / 6.4

Local Drainage Components and Subbasins

The site grade in the STP Units 3 and 4 power block area will range from 11.1 m (36.6 ft) MSL in the center to 9.8 m (32 ft) MSL at the corner, with an approximate gradient of 0.4 percent toward the corners. The power block and the UHS will be located inside the security perimeter. Local East and West Channels will collect runoff within the security perimeter and will discharge to the north across through narrow grated openings in concrete security barriers and underground culverts across security fences. These channels join the Main Drainage Channel (MDC) that runs from east to west north of the STP Units 3 and 4 site.

Catch basins will collect runoff from the STP Unit 3 power block area and direct the discharge to the East Channel by connecting drainage pipes. Similarly, runoff from the STP Unit 4 power block area will flow to the West Channel, which also collects runoff from the UHS area. Runoff from the switchyard of STP Units 1 and 2 will flow to the MDC, which also collects runoff from an area bounded by FM 521 to the north.

The MDC flows west parallel to the security barriers north of STP Units 3 and 4, then turns southwest near the northwest corner of the security barrier, and continues flowing southwest before joining the LRS. A little upstream of the west access road, the MDC and LRS are connected by a link channel. At approximately 152 m (500 ft) south of the link channel, both the MDC and LRS cross the west access road via separate culverts.

Using USGS topographic maps, aerial surveys, and locations of roads and barriers, the applicant divides the site drainage area into seven subbasins: North1 (3.797 km^2 [1.466 mi^2]), North2 (0.772 km^2 [0.298 mi^2]), North3 (0.458 km^2 [0.177 mi^2]), PBN1 (0.826 km^2 [0.319 mi^2]), PBW1 (0.127 km^2 [0.049 mi^2]), PBW (0.350 km^2 [0.135 mi^2]), and PBE (0.231 km^2 [0.089 mi^2]).

Peak Discharges

The applicant used the USACE Hydrologic Engineering Center Hydrologic Modeling System (HEC-HMS) computer model to determine peak discharges for the seven subbasins. The applicant assumed that the whole site drainage is impervious at the start of and during the local PMP event. The applicant estimated the times of concentration for the subbasins using the U.S. Natural Resources Conservation Service (NRCS) recommendations (NRCS 1986). To account for nonlinear effects during extreme floods, the estimated times of concentration were reduced by 25 percent, as recommended in USACE Engineering Manual EM-1110-2-1417 (USACE, 1994). The applicant estimated the lag time as 60 percent of the corresponding time of concentration described by the USACE (2006).

The LRS passes under FM 521 through pipe culverts. The applicant assumed that during the local PMP event, runoff upstream of FM 521 will accumulate while there will be some runoff contribution to LRS via the pipe culverts. After the runoff accumulation results in overtopping FM 521, more runoff from the north of FM 521 will contribute to the LRS.

The applicant sets up the site hydrologic model in HEC-HMS as shown in the hydrologic diagram (FSAR Figure 2.4S.2-6) using the subbasin areas, local PMP intensities, lag times, a runoff curve number of 98 representing impervious areas, and the NRCS dimensionless unit hydrograph option. FSAR Table 2.4S.2-2 shows the subbasin properties, peak discharges, and times to peak. The applicant uses the site hydrologic model to compute the runoff hydrograph during the local PMP event. Because of longer lag times, the storage of runoff upstream of FM 521, and the subsequent overtopping of FM 521, the combined peak discharge from subbasins North1 and North2 occurs at hour 6:25 at the upstream boundary of the LRS. Therefore, the peak discharge into the LRS at its confluence with the MDC occurs much later than the flood peak in the MDC, which also receives runoff from subbasins PBN1, PBE, PBW, and PBW1. FSAR Table 2.4S.2-6 shows the peak discharge at various locations within the site drainage area. The applicant estimates that the peak discharge at the outfall where the LRS and the MDC meet is 279 m^3/s (9,852 cfs).

Hydraulic Model Setup

The applicant estimates the maximum water surface elevation during the local site flooding under a local PMP event using the USACE Hydrologic Engineering Center-River Analysis System (HEC-RAS) model (USACE 2005). The applicant develops the cross sections at several places on the LRS, MDC, and East and West Channels for inclusion in the hydraulic model (FSAR Figure 2.4S.2-7). The applicant obtains the bottom elevations, longitudinal slopes, and side slopes of the channels from site design details (the MDC and East and West Channels) or from an aerial survey (the LRS).

The applicant inputs the inflow discharges in the HEC-RAS model from estimated HEC-HMS discharge hydrographs. In the HEC-HMS computations, peak discharge at the outflow of the site area occurs within 25 minutes of the peak discharges for subbasins PBE, PBW, PBW1, and PBN1. Therefore, the applicant conservatively assumes that peak discharge in each of these individual subbasins coincides with the peak discharge at the outlet (hour 3:35), which also makes the peak discharge into the HEC-RAS model greater than that computed by the HEC-HMS model. In contrast, the peak discharge into the LRS occurs much later (hour 6:25). Therefore, the applicant specifies the input discharge to the LRS as the discharge at hour 3:35 from its HEC-HMS discharge hydrograph. The applicant distributes the peak discharge from each subbasin to the corresponding channel reach using a proportioning approach. The peak discharge for the most upstream cross section in a channel reach is proportional to the contributing area upstream of that reach. The applicant obtains the peak discharges for the remaining cross sections by subtracting the peak discharge at the most upstream cross section from the peak discharge for the whole basin, and then dividing the remainder by the number of remaining cross sections.

The applicant assumes the pipe culverts through which the MDC and LRS cross the west access road to be completely blocked during the local PMP event. Therefore, the applicant models the west access road as an in-line weir in the HEC-RAS. The applicant estimates the width and breadth of the weir from an aerial survey using a weir coefficient of 2.6. The applicant uses Manning's n values recommended by Chow (1959).

The applicant uses the HEC-RAS model to simulate steady-state, subcritical flow conditions in the site drainage area. A sensitivity analysis of the model indicates that the flow over the weir at the west access road is controlled by upstream boundary conditions if the water surface elevation downstream of the weir is less than 10.4 m (34 ft) MSL. It is unlikely that water surface elevations downstream of the west access road will exceed 10.4 m (34 ft) MSL, because most of the runoff upstream of the weir is intercepted by the west access road. Therefore, the applicant uses a constant water level of 10.4 m (34 ft) MSL as the downstream boundary condition in the HEC-RAS simulation.

Flood Elevations

The applicant estimates the maximum water surface elevation in the power block area to be 11.2 m (36.6 ft) MSL from the HEC-RAS simulation. Because this water surface elevation is less than that from the breach of the main cooling reservoir embankment, flood from a local PMP event on the site does not result in the design-basis flood.

The applicant incorporates by reference Section 2.1 of the certified ABWR DCD referenced in Appendix A to 10 CFR Part 52. Due to higher-than-site-grade flood levels from the postulated breach of the main cooling reservoir, the applicant identifies a departure, STP DEP T1 5.0-1, from the certified design. The applicant provides information to address COL License Information Item 2.14 from the generic DCD.

NRC Staff's Technical Evaluation

Probable Maximum Precipitation Depths

The staff reviewed the description of the applicant's local PMP. The staff determined that the applicant's method is acceptable because SRP Section 2.4.2 recommends that method. In an independent analysis, the staff estimated the local PMP from HMR 51 and 52 and obtained values that closely match the applicant's values in FSAR Section 2.4S.2. Therefore, the staff agreed with the applicant's local PMP depth estimates.

FSAR Table 2.0-2 shows that the precipitation site characteristic at the STP site, defined by the local PMP rate, is 50.3 cm/hr (19.8 in./hr), which exceeds the ABWR DCD envelope value of 49.3 cm/hr (19.4 in./hr). The staff issued **RAI 02.04.02-1** requesting the applicant to discuss the additional load on safety-related SSCs as a result of this exceedance and to demonstrate that sufficient safety margins exist in the design of these SSCs.

In a letter dated June 12, 2008 (ML081710126), the applicant's response to **RAI 02.04.02-1** states that the reactor building, the control building, and two RSW pump houses are the only safety-related SSCs that will be affected by the local PMP. The applicant also states in FSAR Tier 2, Subsections 3H.1.4.2 and 3H.2.4.2.5, that the roofs of the safety-related SSCs are either designed without parapets or with scuppers. The applicant adds that these roof designs meet the provisions of RG 1.102.

The staff reviewed the applicant's response and determined that the safety-related SSCs for Units 3 and 4 will be designed so that either their roofs have no parapets or the roofs are equipped with scuppers. The staff determined that the slight exceedance of 2.1 percent in the design-basis roof load due to the local PMP site characteristic would not result in excessive ponding, because the scuppers would assist in draining ponded water away from the roofs of safety-related SSCs. Therefore RAI 02.04.02-1 is considered closed.

Local Drainage Components and Subbasins

The staff reviewed the description of site drainage components and subbasins the applicant includes in FSAR Section 2.4S.2. The staff determined that this description matches the staff's observations of the site during safety and environmental site visits. The staff agreed, therefore, with the applicant's description of local drainage components and subbasins.

Peak Discharges

The applicant selected the USACE HEC-HMS model to estimate peak discharges in the site drainage area under a local PMP event. The staff agreed that HEC-HMS is an appropriate computer model to apply when determining the peak discharge from local site drainages. This model is one of the USACE models recommended in SRP Sections 2.4.3 and 2.4.4.

The applicant provides the HEC-HMS input and output files in electronic format. The staff reviewed the applicant's modeling work and determined that these data are sufficient to adequately estimate peak discharges.

The staff issued **RAI 02.04.02-3** requesting the applicant to discuss (a) flood magnitude and timing; (b) the effect on water levels in the power block area; and (c) the effect of the 10.4-m (34-ft) MSL constant water-level boundary condition in the HEC-RAS simulation, if local access road FM 521 does not act like a barrier and flood runoff from the North1 and North2 subbasins is not significantly lagged. The staff also asked the applicant to (d) justify using a 6-hour PMP rather than a PMP value of a shorter duration and more intensity to obtain peak PMF water surface elevations in the power block area; and (e) specify in the FSAR the point where the peak floodwater surface elevation is simulated within the power block area. The applicant's responses to subparts (a) and (d) are relevant to the discussion in this subsection. The responses to the other subparts are described in the "Flood Elevations" subsection below.

In a letter dated August 12, 2008 (ML091811141), the applicant's response to **RAI 02.04.02-3** refers to the local PMF analysis in the FSAR as the COL application base case. The applicant provides two modeling scenarios with respect to how FM 521 affects the peak discharges near the power block area. In the first scenario, the applicant assumes that FM 521 will not act as a barrier to runoff generated in the North1 and North2 subbasins, and the combined runoff will discharge at the top of the LRS reach. In the second scenario, the applicant assumes that FM 521 will not act as a flow barrier at all. The applicant therefore concludes that the north subbasin of the local site drainage area will consist of a single, larger subbasin that will include the drainage areas of North1, 2, and 3 subbasins. This single, larger north subbasin will discharge directly at the outfall location (the junction where the LRS and the MDC meet).

The applicant states in the response to **RAI 02.04.02-3** that the first of the two scenarios resulted in a higher peak discharge at the bottom of the LRS (approximately 275.1 m^3/s [9,715 cfs]) than the COL application base case (approximately 217.7 m^3/s [7,687 cfs]). The peak discharge also occurred earlier in the first scenario—5 hours and 25 minutes after the beginning of the local PMP storm—compared to the COL application base case timing of 6 hours and 25 minutes after the beginning of the local PMP storm. The applicant also reports that the predicted peak discharge at the outlet for the first scenario was approximately 324.5 m^3/s (11,460 cfs), which is greater than the 279 m^3/s (9,852 cfs) for the COL application base case and occurs at nearly the same time (3 hours and 35 minutes compared to 3 hours and 40 minutes, respectively, after the beginning of the local PMP storm).

The staff reviewed the applicant's responses and found that the applicant's modeled scenarios represent a reasonable sensitivity analysis for the COL application base case results in the FSAR. The staff independently performed the HEC-HMS simulations using the applicant's input files and confirmed that the applicant's reported simulated peak discharges are accurate. The applicant states that the local intense precipitation data used to estimate discharges near the power block area consist of several shorter duration rainfall depths corresponding to 5 minutes, 15 minutes, 1 hour, 2 hours, 3 hours, and 6 hours. The applicant therefore concludes that the effects of more intense precipitation corresponding to durations shorter than 6 hours are captured by the local PMP distribution.

The staff reviewed the applicant's response regarding the precipitation distribution used to estimate flood discharges during the local intense precipitation event. The staff agreed with the applicant's statement that the higher expected intensity of precipitation for events with a shorter duration is represented within the distribution used by the applicant.

Hydraulic Model Setup

The applicant selects the USACE HEC-RAS model to simulate the hydraulics of flooding in site drainage channels and the adjacent LRS during the local PMP event. Because this model is one of the recommended models in the SRP, the staff determined that the HEC-RAS is an appropriate model to apply to the simulation of channel hydraulics during the local PMP event.

The staff reviewed the applicant's modeling work and determined that the data were sufficient for the staff's review and subsequent confirmatory analysis. The staff used the applicant's model to carry out an independent confirmatory analysis of the peak floodwater elevations in the site drainage area under the local PMP event.

Flood Elevations

The applicant uses the USACE HEC-RAS model to estimate flood elevations at the site during the local PMP event. The staff determined that the HEC-RAS is an appropriate model for this purpose as this model was supported and widely used by the US Army Corps of Engineers.

The staff issued RAI 02.04.02-2 requesting the applicant to provide input and output files used in the HEC-RAS simulations. In attachments to a letter dated June 12, 2008 (ML081710126), the applicant provided the HEC-RAS input and output files in electronic format.

The applicant responded to RAI 02.04.02-3 in a letter dated August 12, 2008 (ML091811141). The applicant's responses to subparts (b), (c), and (e) are relevant to the discussions in this subsection.

The applicant states that steady flow routing in the HEC-RAS was used to estimate water surface elevations near the power block area. The applicant specified inflows into the HEC-RAS model cross sections using the time when the peak discharge occurred at the outfall—3 hours and 40 minutes after the beginning of the storm for the reaches, the LRS, and North3—which had a peak discharge time significantly different from 3 hours and 40 minutes. The applicant used peak discharges for the other reaches regardless of their timing. The applicant notes that this approach is similar to that used in the COL application base case. The resulting peak discharge at the outfall was about 376 m^3/s (13,293 cfs), or approximately 20 percent higher than the 313.8 m^3/s (11,080 cfs) used in the COL application base case.

The applicant states that for the first scenario simulation, the maximum water surface elevation near the power block area was 11.22 m (36.8 ft) MSL. The maximum water surface elevation occurred in the East Channel at three locations: the most upstream river station and two cross sections near the proposed location of the Unit 3 reactor building. The maximum simulated water surface elevation is slightly higher than the 11.16 m (36.6 ft) MSL in the COL application base-case simulation. The applicant also states that the higher water surface elevation is a result of the conservative assumption related to the effect of the postulated FM 521 breach, which ignores any attenuation of flood peaks due to backwater effects at the breach. The assumption also ignores any diversion of flood flow away from the LRS and MDC following the FM 521 breach.

The applicant states that the peak elevation of the floodwater surface of 11.16 m (36.6 ft) MSL will occur in the East Channel within the protected area boundary and will affect the safety-related reactor and control buildings. The applicant also states that the peak elevation of the floodwater surface along the entire West Channel will be about 11.1 m (36.4 ft) MSL. Therefore, the applicant conservatively assumes that the entire power block area will be affected by a maximum elevation of the floodwater surface of 11.16 m (36.6 ft) MSL as a result of local intense precipitation. The applicant has updated the FSAR to state the maximum water surface elevation within the power block area of STP Units 3 and 4.

The staff reviewed the applicant's response and agreed that the model represents a conservative scenario in terms of flooding near the power block area. Based on the minor increase in the maximum elevation of the floodwater surface under conservative assumptions regarding the FM 521 breach, the staff determined that the maximum elevation of the floodwater surface near the power block area would be less than the design-basis elevation of the floodwater surface resulting from the main cooling reservoir breach. The staff concluded that flooding near the power block area resulting from the local intense precipitation event is not the controlling flood scenario at the STP Units 3 and 4 site. Therefore RAI 02.04.02-2 and RAI 02.04.02-3 are closed. In response to RAI 02.04.02-3, the applicant proposed to revise the first paragraph of FSAR Section 2.4S.2.3.5 to specify the spot at which the peak flooding level was simulated. The FSAR update proposed in the response of RAI 02.04.02-3 will be tracked as Confirmatory Item 02.04.02-1.

The staff needed more detailed information on the HEC-RAS model to understand the procedure used to evaluate its conservatism..The staff issued RAI 02.04.02-4 requesting the applicant to elaborate on the following statements in FSAR Subsection 2.4S.2.3.4, page 2.4S.2-8: "The peak discharge obtained for a subbasin in HEC-HMS was first distributed to the most upstream cross section of a stream reach in the HEC-RAS in proportion to the area contributing to that cross section and the total area of the subbasin. The remaining portion of the peak discharge is then distributed equally among the remaining cross sections within the receiving channel reach."

In a letter dated July 9, 2008 (ML081960070), the applicant's response to RAI 02.04.02-4 states that the discharges simulated by the HEC-HMS for each of the subbasins that drain into the HEC-RAS channel reaches were distributed among the cross sections within the reach based on the drainage area upstream of the respective cross section. The applicant also states that for the North3 subbasin, which drains into the LRS, the selected flood flow from North3 at the time of peak discharge at the outlet was divided among the 11 cross sections of the LRS. The applicant specifies the discharge for the most upstream cross section of the LRS as the inflow from the storage element at its upstream end, which receives inflows from the North1 and North2 subbasins plus one-eleventh of the flood discharge from the North3 subbasin. The applicant notes that each downstream cross section of the HEC-RAS LRS reach receives an additional one-eleventh of the flood discharge from North3. The applicant uses a similar approach to distribute the flood discharge from subbasin PBN1. The applicant states that approximately 0.26 km^2 (0.1 mi^2) of the PBN1 drainage area directly discharges into the most upstream cross section of the HEC-RAS MDC reach. To estimate this discharge, the applicant multiplies the peak discharge from PBN1 by the ratio of 0.26 km^2 (0.1 mi^2) to the total drainage area of PBN1, 0.83 km^2 (0.319 mi^2). The applicant distributes the rest of the peak discharge from PBN1 among the 27 cross sections of the MDC reach.

The staff reviewed the applicant's response and concluded that it is a reasonable approach for specifying discharges from adjacent drainage areas into each of the HEC-RAS cross sections used in the simulation of the elevation of the floodwater surface. The staff determined, therefore, that the applicant's response is satisfactory. Therefore RAI 02.04.02-4 is considered closed.

2.4S.2.5 Post Combined License Activities

There are no post-COL activities related to this subsection.

2.4S.2.6 Conclusion

The staff reviewed the applicant's submittals in FSAR Section 2.4S.2 and in response to the RAIs. Based on this review, the staff determined that the applicant has appropriately described the flood history, flood causal mechanisms, local intense precipitation, and the estimation of the local PMF near the STP site and no outstanding information is expected to be addressed in this section. The staff found that the applicant has considered the appropriate site phenomena for establishing the site flood causal mechanisms. The staff accepted the methodologies used to determine the local intense precipitation, flood causal mechanisms, and controlling flood mechanisms. Accordingly, the staff concluded that the use of these methodologies results in site characteristics with a margin sufficient for the limited accuracy, quantity, and period of time in which the data were accumulated.

The staff reviewed the applicant's estimated local intense precipitation rates and found that the applicant's estimated values closely match those estimated independently by the staff. The staff also found, based on independent confirmatory analyses, that the applicant had used a conservative approach to estimate the flood levels at and near the power block area of proposed STP Units 3 and 4. In conclusion, the applicant has provided sufficient information for satisfying 10 CFR Part 52 and 10 CFR Part 100. The information addressing COL License Information Item 2.14 is adequate and acceptable.

2.4S.2.7 References

American Nuclear Society, "Determining Design Basis Flooding at Power Reactor Sites," *Historical Technical Reference*, American Nuclear Society ANSI/ANS-2.8, July 1992.

Chow, V. T., *Open-Channel Hydraulics*, McGraw-Hill Book Co., New York, NY, 1959.

Halff Associates, Inc., "1992 Colorado River Flood Damage Evaluation Project, Final Report, Phase I, Volume I and Volume II," prepared for the Lower Colorado River Authority and Fort Worth District Corps of Engineers, July 2002.

Hansen, E.M., L.C. Schreiner, and J.F. Miller, "Application of Probable Maximum Precipitation Estimates, United States East of the 105th Meridian," NOAA Hydrometeorological Report 52, National Weather Service, Washington, DC, 1982.

Natural Resources Conservation Service, "Urban Hydrology for Small Watersheds," Technical Release 55, U.S. Department of Agriculture, Natural Resources Conservation Service, 2nd Ed., revised June 1986, update of Appendix A, January 1999.

Pararas-Carayannis, G., "Verification Study of a Bathystrophic Storm Surge Model," U.S. Army Corps of Engineers Technical Memorandum No. 50, Coastal Engineering Research Center, Fort Belvoir, VA, May 1975.

Schreiner L.C., and J.T. Riedel, "Probable Maximum Precipitation Estimates, United States East of the 105th Meridian," NOAA Hydrometeorological Report 51, National Weather Service, Washington, DC, 1978.

South Texas Nuclear Operating Company, "South Texas Project Combined License Application," Revision 0, Part 2, Final Safety Analysis Report, 2007.

U.S. Army Corps of Engineers, "Flood-Runoff Analysis," U.S. Army Corps of Engineers Engineer Manual 1110-2-1417, Washington, DC, August 1994.

U.S. Army Corps of Engineers, "HEC-RAS, River Analysis System, User's Manual," Version 3.1.3, Hydrologic Engineering Center, 2005.

U.S. Army Corps of Engineers, "Hydrologic Modeling System, HEC-HMS," User's Manual, Version 3.1.0, U.S. Army Corps of Engineers, Hydrologic Engineering Center, Washington, DC, November 2006.

2.4S.3 Probable Maximum Flood (PMF) on Streams and Rivers

2.4S.3.1 Introduction

This section of the FSAR describes the hydrological site characteristics affecting any potential hazard to the plant's safety-related facilities as a result of the effect of the PMF on streams and rivers.

Section 2.4S.3 of this SER provides a review of the following specific areas: (1) regional probable maximum precipitation and precipitation losses, (2) runoff and stream course models, (3) PMF, (4) consideration of other site-related evaluation criteria, and (5) any additional information requirements prescribed in the "Contents of Application" sections of the applicable subparts of 10 CFR Part 52.

2.4S.3.2 Summary of Application

In Section 2.4S.3, the applicant addresses the information about site-specific PMFs on streams and rivers. In addition, in this section, the applicant provides site-specific supplemental information to address COL License Information Item 2.15 identified in DCD Tier, Revision 4, Section 2.3.

COL License Information Item

- COL License Information Item 2.15 Probable Maximum Flood on Streams and Rivers

COL License Information Item 2.15 requires COL applicants to provide the basis for determining the protection of safety-related structures against a PMF.

2.4S.3.3 Regulatory Basis

The associated acceptance criteria for identifying a PMF and flood design considerations are in Section 2.4.3 of NUREG-0800.

The applicable regulatory requirements for identifying probable maximum flooding on streams and rivers are as follows:

- 10 CFR Part 100, as it relates to identifying and evaluating hydrological features of the site. The requirements to consider physical site characteristics in site evaluations are specified in 10 CFR 100.20(c).

- 10 CFR 100.23(d)(3), as it sets forth the criteria to determine the siting factors for plant design bases with respect to seismically induced floods and water waves at the site.

- 10 CFR 52.79(a)(1)(iii), as it relates to identifying hydrologic site characteristics with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

In addition, the staff used the regulatory positions of the following regulatory guides for the identified acceptance criteria:

- RG 1.27, "Ultimate Heat Sink for Nuclear Power Plants"
- RG 1.59, "Design Basis Floods for Nuclear Power Plants," as supplemented by best current practices

2.4S.3.4 Technical Evaluation

The staff reviewed the information in Section 2.4S.3 of the STP Units 3 and 4 COL FSAR. The staff's review confirmed that the information in the application addresses the relevant information related to the PMF. The staff's technical review of this application included an independent review of the applicant's information in the FSAR and in the responses to the RAIs. The staff supplemented this information with other publicly available sources of data.

This section describes the staff's evaluation of the technical information in FSAR Section 2.4S.2.

COL License Information Item

- COL License Information Item 2.15 Probable Maximum Flood on Streams and Rivers

The staff reviewed site-specific information related to a PMF and the potential for flooding at the plant site, including the effects of local intense precipitation.

2.4S.3.4.1 Probable Maximum Precipitation

Information Submitted by the Applicant

The applicant estimates the PMP over the Colorado River Basin below the Mansfield Dam in FSAR Section 2.4S.3.1. The applicant's analysis is based on the PMP established in several studies, namely:

- Updated Final Safety Analysis Report (UFSAR) for STP Units 1 and 2 (STPEGS 2006)
- A PMF analysis conducted for Mansfield Dam (USBR, 1985, 1989, 2003, 2007; Goodson and Associates, 1990)
- A dam safety analysis for the Lower Colorado River (Freese and Nichols. 1992)
- A flood damage study for the Lower Colorado River (Halff Associates, Inc.. 2002).

The applicant follows the procedures described in National Oceanic and Atmospheric Administration (NOAA) NWS HMRs 51 and 52 (Schreiner and Riedel, 1978; Hansen et al., 1982) to obtain the spatial distribution of the PMP within the basin. The applicant estimates the critical centering of the PMP storm pattern that would produce the greatest volume of precipitation within the drainage basin. The applicant analyzes two different storm pattern orientations for the drainage basin to derive the most critical PMF hydrographs near the STP Units 3 and 4 site.

Previous studies (Halff Associates, Inc., 2002 and STPEGS, 2006) used a 96-hour PMP storm duration because the peak discharge from the Upper Colorado River Basin reaches Mansfield Dam and the peak discharge from areas in the Lower Colorado River Basin reaches Wharton by the end of the storm event. The applicant also selects the 96-hour duration storm as the PMP hyetograph for estimating the PMF at the STP Units 3 and 4 site.

The applicant notes that previous studies (USBR 1985) demonstrate that the largest floods in the Colorado River Basin result from frequent and intense summer rainfall events. Therefore, the applicant does not consider snowmelt or rainfall on antecedent snowpack in estimating the PMF in the Lower Colorado River Basin.

NRC Staff's Technical Evaluation

The staff reviewed the applicant's information in FSAR Section 2.4S.3 (STPNOC, 2007). The applicant uses NOAA NWS HMR 51 and 52 to estimate the PMP in the Lower Colorado River Basin. The staff verified the 6-, 12-, 24-, 48-, and 72-hour PMP depths from HMR 51 for the subbasin CC-06 which was identified previously (Halff Associates, Inc. 2002) as the center of the critical storm that produces the largest flow rate at Bay City. Based on this review, the staff determined that the applicant's estimates are reasonable.

HMR 51 provides PMP depths for durations up to 72 hours only. The staff reviewed the applicant's method for extrapolating the PMP depths for the 96-hour duration. Generally, the rate of increase in the precipitation depth reduces as the duration of precipitation increases. Therefore, the slope of the depth-duration relationship becomes flatter with increasing duration. For the CC-06 subbasin, the incremental PMP depths for the second day (hours 24 to 48) and the third day (hours 72 to 96) are 13.1 and 7.9 cm (5.2 and 3.1 in.), respectively. The applicant's extrapolation to the fourth day (hours 72 to 96) resulted in an additional 6.6 cm (2.6 in.) of precipitation depth. The staff's review determined that the applicant's method for extrapolating the PMP depth is reasonable, because the relationship between the incremental PMP depth and duration shows a persistent decreasing trend.

The applicant uses the HMR 52 spatial distribution of the PMP pattern. HMR 52 recommends PMP estimation for basin areas equal to or less than 51,780 km^2 (20,000 mi^2). The applicant uses two separate storm pattern for the 57,721 km^2 (22,682 mi^2)-Lower Colorado River Basin-one for the upper and the other for the lower part of the basin. HMR 52 recommends using a single PMP storm pattern, but this approach for the 57,721 km^2 (22,682 mi^2)-Lower Colorado River Basin would result in less precipitation compared to the applicant's approach that uses two separate storm patterns—one for the upper and the other for the lower part of the basin, both of which are smaller in area than the whole basin and therefore would result in PMP intensities that are higher than a hypothetical 57,721 km^2 (22,682 mi^2)-PMP. The use of the two storm patterns would result in additional precipitation falling on the remote areas of an elongated basin. Therefore, the staff accepts the applicant's two storm-pattern approach because it is more conservative and would result in a larger flood at the STP Units 3 and 4 site.

Because snow accumulations in the Lower Colorado River Basin occur infrequently, the staff agreed with the applicant that a snowmelt or rain-on-snow event is unlikely to produce a PMF in the Lower Colorado River Basin (see staff's evaluation in Section 2.4S.3.4.3 below).

2.4S.3.4.2 Precipitation Losses

Information Submitted by Applicant

The applicant discusses precipitation losses in FSAR Section 2.4S.3.2 and Subsection 2.4S.3.4.2.1 (STPNOC, 2007). The applicant assumes no initial losses in the HEC-HMS modeling. The applicant uses guidelines from the Federal Energy Regulatory Commission (FERC, 2001) to specify a uniform continuing loss rate of 0.13 cm/hr (0.05 in./hr).

NRC Staff's Technical Evaluation

The staff issued **RAI 02.04.03-7(c)** requesting the applicant to discuss how the constant precipitation loss rate of 0.13 cm/hr (0.05 in./hr), adopted for the PMF study, is conservative, as the applicant states in FSAR, Subsection 2.4S.3.4.2.1. In a letter dated July 2, 2008 (ML081890239), the applicant's response to **RAI 02.04.03-7(c)** states that a uniform continuing loss rate of 0.13 cm/hr (0.05 in./hr)was used to estimate the PMF, and that this value is the minimum range recommended by FERC (2001). The applicant has updated the discussion of precipitation losses in the FSAR. The staff reviewed the applicant's reference and agreed that the precipitation loss rate used in the PMF study is conservative, therefore RAI 02.04.03-7(c) is closed.

2.4S.3.4.3 Runoff and Stream Course Models

Information Submitted by Applicant

The applicant discusses the runoff model in FSAR Section 2.4S.3.3 (STPNOC 2007). Halff Associates, Inc. (2002) developed the HEC-HMS model that included 80 subbasins in the Lower Colorado River Basin extending from below the Mansfield Dam to Bay City. Halff Associates, Inc., calibrated this model to simulate floods up to 100-year storm events. The applicant modifies the Halff model conservatively by decreasing runoff lag times by 25 percent, as recommended by the USACE (1994), and by using modified rating curves for the channel reaches to account for larger flows during the PMF event.

NRC Staff's Technical Evaluation

The staff issued **RAI 02.04.03-7(b)** requesting the applicant to provide details about how it reached the following conclusion: "snow melt and antecedent snow pack are not a factor in the production of floods at the STP 3 & 4 site," in FSAR, Section 2.4S.3.1. In a letter dated July 2, 2008, the applicant's response to RAI 02.04.03-7(b) (ML081890239), states that previous studies of PMF in the Colorado River Basin have noted that frequent and intense rainfall events occurring simultaneously over several subbasins produced the largest recorded floods in the river. The rainfall distribution during a year in the Colorado River Basin has two peaks, one in May and one in September. Spring rainfall events are produced by convective thunderstorms, while late summer or early fall rainfalls are associated with tropical cyclones. The applicant also states that, because the climate in the Colorado River Basin is not suitable for appreciable snowpack development, snow melt or rainfall on antecedent snowpack will not produce a PMF in the Lower Colorado River Basin near the STP Units 3 and 4 site.

The staff reviewed the applicant's information and determined that the hydrometeorological characteristics of the Colorado River Basin, especially the Lower Colorado River Basin, are not suitable for the development of large snowpacks or wintertime floods. The staff concluded, therefore, that snow melt and rainfall on antecedent snowpack would not cause a PMF at the site.

The staff reviewed the above runoff and stream course models used by the applicant, and concluded that the applicant has appropriately selected numerical models and has used appropriate data sets and parameter values to represent the hydrologic characteristics of the Lower Colorado River Basin. Therefore, RAI 02.04.03-7(b) is considered closed.

2.4S.3.4.4 Probable Maximum Flood Flow

Information Submitted by Applicant

The applicant discusses the estimation of PMF flow in FSAR Section 2.4S.3.4 along with details of the previous studies. FSAR Table 2.4S.3-1 summarizes estimates of peak flow at Mansfield Dam from different studies for comparison (STPNOC, 2007). The applicant bases the PMF scenarios at the STP Units 3 and 4 site on the PMF scenarios considered for STP Units 1 and 2. The applicant eliminated some of the previous scenarios because of abandoned plans to build the Shaw Bend Dam on the Lower Colorado River. The three remaining scenarios are as follows:

- Scenario 1: A PMF for the area between Mansfield Dam and Bay City combined with a 3-day antecedent storm equal to 40 percent of the PMP event occurring over the same area 3 days before the PMF event, plus the Mansfield Dam release and the base flow at Bay City.

- Scenario 2: A PMF for the area above Mansfield Dam resulting from a PMP storm in the drainage area from Lake O.H. Ivie to Mansfield Dam, plus a sequential storm equal to 40 percent of the PMP event occurring over the drainage area between Bay City and Mansfield Dam 3 days after the PMP storm upstream of Mansfield Dam combined with the base flow at Bay City.

- Scenario 3: A PMF for the entire Lower Colorado River Basin area between Lake O.H. Ivie and Bay City, with an antecedent Standard Project Storm for the same area added to the base flow at Bay City (Halff Associates, Inc. 2002).

The applicant uses the scenario that produces the highest PMF discharge as the most critical. Based on the previous studies and additional hydrodynamic modeling analyses, the applicant concludes that Scenario 1 is the critical scenario, and uses it to establish the PMF peak discharge of 39,571 m^3/s (1,397,432 cfs) (FSAR Section 2.4S.3.4.3) (STPNOC 2007).

NRC Staff's Technical Evaluation

The staff reviewed the applicant's modeling approach for assessing the regional PMF in the Lower Colorado River Basin. The staff found that the applicant's selection of the numerical model and the associated parameters are appropriate and that the basin representation within the model is acceptable. As discussed in the Subsection 2.4S.3.4.3 of this SER, the staff noted that the applicant uses conservative assumptions and input parameters such as rainfall distributions and loss rates. Therefore, the staff concluded that the applicant's estimate of the

PMF discharge into the Colorado River near the STP Units 3 and 4 site is appropriate and conservative.

2.4S.3.4.5 Water Level Determinations

Information Submitted by Applicant

The applicant discusses water-level determinations in FSAR Section 2.4S.3.5 (STPNOC 2007). To put the estimated flood level in context, the applicant uses the following elevations:

- the site nominal grade for safety-related facilities: 10.4 m (34.0 ft) MSL (FSAR Section 2.4S.4)
- the site safety-related entrance slab elevation: 10.7 m (35.0 ft) MSL (FSAR Section 2.4S.4)
- the referenced ABWR DCD site flood level is 0.3 m (1 ft) below the nominal grade: 10.1 m (33.0 ft) MSL
- all ventilation openings of safety-related buildings are located at or above 12.2 m (40 ft) MSL (FSAR Subsection 2.4S.4.3.2).

The applicant uses the HEC-RAS computer program to determine the flood level at the site corresponding to the PMF peak discharge. The Halff study (Halff Associates, Inc., 2002) developed and calibrated the HEC-RAS steady-state model for simulating the floods in the Lower Colorado River Basin. The applicant uses the same model but changes some parameter values from the Halff study. For example, the applicant increases the Manning's roughness coefficients by 20 percent from the calibrated values used in the Halff study to account for the increased roughness in the overbank and floodplain areas where the PMF discharge is expected to occur. The applicant states that, because the calibrated Manning's roughness coefficients cannot be used for a hypothetical high-magnitude flood such as a PMF event, the applicant increases the roughness coefficients based on the published recommendations (Smith, 1992). The applicant also expands the width of the river cross sections to simulate adequately the PMF discharges on the floodplain.

The applicant sets the downstream water surface elevation boundary condition to a normal flow depth. Using the HEC-RAS model with conservatively higher roughness coefficient values of the floodplain than those used in the Halff Associates study, the applicant determines the normal water depth at the downstream boundary to be 5.3 m (17.5 ft) NAVD88. The applicant's estimates of the normal depth with the Halff study roughness value is 4.9 m (16.2 ft) NAVD88. Using the HEC-RAS model with the above downstream boundary condition and the PMF inflow into the basin, the applicant estimates a water surface elevation at the site of 8.0 m (26.1 ft) NAVD88, which is approximately 2.7 m (9 ft) lower than the STP Units 3 and 4 site grade (FSAR Figure 2.4S.3-2). The applicant states that the above PMF flood-level estimate is higher (thus more conservative) than the one estimated with the Halff study roughness values of the floodplain.

NRC Staff's Technical Evaluation

The staff issued **RAI 02.04.03-6** requesting the applicant to explain why it states in FSAR Subsection 2.4S.3.5.3.1 that the water level in the Colorado River at the most downstream cross section used in the HEC-RAS model is unaffected by tidal conditions. The applicant's response dated July 9, 2008 (ML081960070), states that under PMF conditions, the discharge into the Colorado River will be 39,570 m^3/s (1,397,432 cfs) at the downstream boundary, and the corresponding normal depth of flow will be an estimated 5.3 m (17.5 ft) NAVD88 or 5.4 m (17.7 ft) NGVD29. The applicant reports the maximum water level recorded at the NOAA tide gauge at Freeport, Texas, as 1.51 m (4.95 ft) MSL. Because the PMF water surface elevation at the normal depth exceeds the maximum tidal level, the applicant concludes that the normal depth at the downstream boundary is the appropriate boundary condition to use in the HEC-RAS model.

The staff reviewed the applicant's response and agreed that the large PMF discharge would occur at a greater depth of flow at the downstream boundary of the HEC-RAS modeling domain and would therefore be unaffected by tidal conditions.

The staff also reviewed the applicant's approach for estimating the elevations of floodwater surface near the STP Units 3 and 4 site during a PMF in the Lower Colorado River Basin. The staff determined that the applicant has appropriately selected the numerical model, HEC-RAS, and its associated parameter values and boundary conditions. The staff also found that the applicant adopts the conservatively estimated flood discharges obtained from the HEC-HMS model. The staff concluded that the applicant has appropriately and conservatively estimated the PMF water surface elevation near the STP Units 3 and 4 site. Therefore, RAI 02.04.03-6 is closed.

2.4S.3.4.6 Coincident Wind Wave Activity

Information Submitted by Applicant

The applicant discusses the coincident wind-wave activity in FSAR Section 2.4S.3.6. The applicant does not estimate the coincident wind-wave activity with PMF because the flood elevations for the upstream dam failure and the main cooling reservoir embankment breach scenarios are estimated to be much higher than that of the regional PMF. The applicant concludes that PMF water surface elevations combined with wind waves will not be the controlling scenario at the STP Units 3 and 4 site.

NRC Staff's Technical Evaluation

The applicant does not provide estimates of wave heights from wind wave activity for the PMF water surface elevations. The staff agreed with the applicant that any wind-wave activity coincident with the PMF in the Colorado River would be smaller than that estimated for the upstream dam-failure scenario and therefore would not exceed the estimated elevation of the floodwater surface for that scenario. The staff concludes that estimating the wind-wave effects coincident with the PMF in the Colorado River near the STP Units 3 and 4 site is not necessary.

The staff reviewed Section FSAR 2.4S.3. The staff's review confirmed that the information in the application addresses the relevant information related to this subsection. The staff's technical review of this application includes the following factors:

- appropriateness of the models used in the flood safety analysis
- reasonableness of the parameters chosen in the modeling
- adequacy of the combinations of flood-causing events
- validity of the applicant's safety conclusions for potential PMF hazards at the site.

The staff determined that the models and methods used by the applicant in FSAR Section 2.4S.3 are currently used in standard engineering practices. HEC-HMS and HEC-RAS are routinely used to estimate historical and hypothetical flood hydrographs and the corresponding water surface elevations in rivers and streams. Therefore, the staff concluded that the applicant has appropriately selected numerical models to estimate the PMF and its corresponding water surface elevation in the Colorado River near the STP Units 3 and 4 site.

The staff reviewed the applicant's selection of parameters for the HEC-HMS and HEC-RAS models. The staff agreed with the applicant's determination that unit hydrograph parameters derived for a smaller rainfall event need adjustments to account for the nonlinear basin response during a PMP event. The staff determined that the applicant's approach, which follows the recommendation of USACE (1994), is acceptable. The staff also agreed with the applicant's selection of the loss-rate parameters in the HEC-HMS analysis. Setting the initial loss rate to zero will maximize the runoff generated from the PMP event and is therefore conservative. The staff determined that the continuing loss rate selected by the applicant is based on recommendations of another Federal agency (FERC, 2001) that estimates the PMF for designing and regulating critical hydroelectric dams. The staff concluded that the applicant has conservatively selected the minimum of the recommended continuing loss rates for the HEC-HMS model and thereby has maximized the produced runoff. The staff therefore found that the applicant's selection of the continuing loss-rate parameter to estimate the PMF in the Lower Colorado River Basin is acceptable. The staff also determined that the subbasin configuration and PMP storm patterns used for the HEC-HMS analysis are acceptable.

The staff reviewed the applicant's approach for specifying the HEC-RAS parameters. Because debris is expected to be carried along with a PMF, and because the PMF is expected to inundate the overbank and floodplain areas that typically have greater roughness than the main channel due to the presence of shrubs, vegetation, and other obstacles, the staff determined that the applicant's approach for increasing Manning's roughness coefficients from their baseline values in the Halff study is appropriate. The staff also determined that the adjusted Manning's roughness coefficients used in the HEC-RAS modeling of the PMF (0.042 for the main channel, 0.054 to 0.06 for the overbank, and 0.102 to 0.114 for the floodplain areas) represent a moderately rough main channel and rough floodplain areas. For example, Chow (1959) recommends that Manning's roughness coefficients should range from 0.025 to 0.060 for major streams with a regular cross section and no boulders and from 0.045 to 0.160 for a floodplain with a medium to dense brush. Therefore, the staff determined that use of these parameters would result in a conservative estimate of the elevation of the floodwater surface at the STP Units 3 and 4 site. The staff concluded, therefore, that the applicant has appropriately selected the model parameters.

The staff reviewed the applicant's use of combined events for flooding in rivers and streams as applied to the Lower Colorado River Basin. The applicant uses three combinations of PMF scenarios to determine the most critical combination of events (see Section 2.4S.3.2.4 of this report). The applicant also uses the base flow in the Colorado River near the site combined with the PMF discharge near the STP Units 3 and 4 site, as recommended in

ANSI/ANS-2.8-1992 (ANS, 1992). The PMF stillwater elevation near the STP Units 3 and 4 site is significantly lower than that resulting from the upstream domino-type dam failures and the main cooling reservoir embankment breach. Therefore, the applicant does not specifically estimate wind waves for the PMF water surface elevations. The staff agreed with the applicant's statement that any wind-wave activity coincident with the PMF in the Colorado River will be smaller than that estimated for the upstream dam-failure scenario and will therefore not exceed the estimated elevation of the floodwater surface for that scenario. The staff concluded that the applicant correctly identifies the combination of events for the PMF in the Lower Colorado River Basin.

Based on the above, the staff also agrees with the applicant's analysis that the PMF in the Lower Colorado River Basin is not the controlling flooding mechanism at the STP Units 3 and 4 site. The upstream dam failure and the main cooling reservoir embankment breach scenarios resulted in higher water surface elevations. The staff describes and reviews these flood scenarios in Section 2.4S.4 of this SER. Therefore, the staff determined that the applicant's conclusions regarding the PMF in the Lower Colorado River Basin are valid.

2.4S.3.5 Post Combined License Activities

There are no post-COL activities related to this subsection.

2.4S.3.6 Conclusion

As described above, the NRC staff reviewed the FSAR to determine the adequacy of the applicant's safety conclusions regarding the regional PMF estimates at the site. The staff determined that the applicant has selected appropriate numerical models, has used data and methods commonly used in engineering practices, has conservatively selected model parameters as suggested by studies of a similar nature routinely performed by other Federal agencies, and has used combinations of events recommended in ANSI/ANS-2.8–1992 for nuclear power plant sites. Therefore, there is no outstanding information required to be addressed in this section of COL FSAR. Accordingly, the staff concluded that the use of these methodologies results in site characteristics with a margin sufficient for the limited accuracy, quantity, and period of time in which the data were accumulated.

As set forth above, the applicant presents and substantiates information relative to the potential for site inundation due to the PMF. The staff reviewed the available information and concluded, for the reasons given above, that the identification and consideration of the PMF in the vicinity of the site and site regions are acceptable.

The staff determined that the applicant's conclusions regarding PMF water surface elevation in the Colorado River near the STP Units 3 and 4 site are acceptable. Therefore, the staff concluded that the identified site characteristics meet the requirements of 10 CFR 52.79 and 10 CFR 100.20(c), with respect to establishing the design basis for SSCs important to safety. The information addressing COL License Information Item 2.15 is adequate and acceptable.

2.4S.3.7 References

American Nuclear Society, "Determining Design Basis Flooding at Power Reactor Sites," ANSI/ANS-2.8-1992, Historical Technical Reference, July 1992.

Chow, V.T, *Open Channel Hydraulics*, McGraw-Hill Book Co., New York, NY, 1959.

Federal Energy Regulatory Commission, “Engineering Guidelines for the Evaluation of Hydropower Projects, Determination of the Probable Maximum Flood,” Chapter VIII, 2001; http://www.ferc.gov/industries/hydropower/safety/guidelines/eng-guide.asp; accessed November 30, 2009.

Freese and Nichols, Inc., “Phase II – Dam Safety Evaluation Project, Task Order B, Volume I,” prepared for the Lower Colorado River Authority, August 1992.

Goodson and Associates Inc., “Civil Engineering Report of Intermediate Examination of Marshall Ford Dam, Colorado River Authority, Texas,” December 1990.

Halff Associates, Inc., “1992 Colorado River Flood Damage Evaluation Project, Final Report, Phase I, Volume I and Volume II,” prepared for the Lower Colorado River Authority and Fort Worth District Corps of Engineers, July 2002.

Hansen, E.M., L.C. Schreiner, and J.F. Miller, "Application of Probable Maximum Precipitation Estimates, United States East of the 105th Meridian," NOAA Hydrometeorological Report 52, National Weather Service, Washington, D.C., 1982.

Schreiner L.C., and J.T. Riedel, "Probable Maximum Precipitation Estimates, United States East of the 105th Meridian," NOAA Hydrometeorological Report 51, National Weather Service, Washington, D.C., 1978.

Smith, C.D., “Reliability of Flood Discharge Estimates: Discussion,” *Canadian Journal of Civil Engineering,* 19:1085–1087, 1992.

South Texas Nuclear Operating Company, “South Texas Project Combined License Application,” Revision 0, Part 2, Final Safety Analysis Report, 2007.

STPEGS, “Updated Final Safety Analysis Report (UFSAR) for Units 1 & 2,” Revision 13, May 1, 2006.

U.S. Army Corps of Engineers, “Flood-Runoff Analysis,” Engineer Manual 1110-2-1417, Department of the Army, Washington, D.C., 1994.

U.S. Bureau of Reclamation, “Probable Maximum Flood, Marshall Ford Dam, Lower Colorado River Project, Texas,” United States Department of the Interior, November 1985.

U.S. Bureau of Reclamation, “SEED Analysis Report - Marshall Ford Dam,” ATC Engineering Consultants Inc., prepared for the United States Department of the Interior, Bureau of Reclamation, Colorado River Project, July 1989.

U.S. Bureau of Reclamation, “Mansfield Dam Comprehensive Facility Review, Highland Lakes Dams, Lower Colorado River Authority,” United States Department of the Interior, Bureau of Reclamation, Technical Service Center, Denver, CO, March 2003; http://www.usbr.gov/dataweb/dams/tx01087.htm; accessed by the applicant on February 20, 2007.

2.4S.4 Potential Dam Failures

2.4S.4.1 Introduction

This section of the FSAR addresses potential dam failures to ensure that any potential hazard to safety-related structures due to failure of onsite, upstream, and downstream water-control structures is considered in the plant design.

This section of the SER presents the staff's review of the estimation of the flood level caused by different dam failures. The specific areas of review are as follows: (1) dam-failure permutations, (2) unsteady flow analysis of potential dam failures, (3) water-level determination, and (4) any additional information requirements prescribed in the "Contents of Application" sections of the applicable subparts of 10 CFR Part 52.

The staff reviewed two postulated dam-failure scenarios: (1) dams on the Colorado River upstream of the STP Units 3 and 4, and (2) the main cooling reservoir embankment breach. The staff identifies that the latter case is found to be the controlling scenario with water-level estimates higher than the bounding design flood level specified in the ABWR DCD, which therefore indicates the need for flood protection.

2.4S.4.2 Summary of Application

In Section 2.4S.4, the applicant addresses the site-specific information about potential dam failures. In addition, in this section, the applicant provides site-specific supplemental information to address COL License Information Items 2.14 and 3.5.

<u>*COL License Information Items*</u>

- COL License Information Item 2.14 Floods

COL License Information Item 2.14 requires COL applicants to provide site-specific information related to historical flooding and the potential flooding at the plant site, including flood history, flood design considerations, and effects of local intense precipitation. This information is provided below.

- COL License Information Item 3.5 Flood Elevation

COL License Information Item 3.5 requires COL applicants to ensure that the design-basis flood elevation for the ABWR standard plant structures will be 30.5 cm (12 in.) below grade. This information is provided below.

2.4S.4.3 Regulatory Basis

The relevant requirements of the Commission regulations for the potential dam failures, and the associated acceptance criteria, are in Section 2.4.4 of NUREG–0800.

The applicable regulatory requirements for identifying the effects of dam failures are as follows:

- 10 CFR Part 100, as it relates to identifying and evaluating hydrological features of the site. The requirement to consider physical site characteristics in site evaluations is specified in 10 CFR 100.20(c).

- 10 CFR 100.23(d)(3), as it sets forth the criteria to determine the siting factors for plant design bases with respect to seismically induced floods and water waves at the site.

- 10 CFR 52.79(a)(1)(iii), as it relates to identifying hydrologic site characteristics with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

In addition, the staff used the regulatory positions of the following regulatory guides for the identified acceptance criteria:

- RG 1.27, "Ultimate Heat Sink for Nuclear Power Plants"

- RG 1.59, "Design Basis Floods for Nuclear Power Plants," as supplemented by best current practices

- RG 1.102, "Flood Protection for Nuclear Power Plants"

2.4S.4.4 Technical Evaluation

The staff reviewed the applicant's information in FSAR Section 2.4S.4. The staff's review confirmed that the information in the application addresses the relevant information related to the potential dam failures. The staff's technical review of this section includes an independent review of the applicant's information in the FSAR and in the responses to the RAIs. The staff supplemented this information with other publicly available sources of data.

This section describes the staff's evaluation of the technical information presented by the applicant in FSAR Section 2.4S.4. This FSAR section considers the following:

- inundation due to offsite river dam failures
- inundation due to a breach of the main cooling reservoir embankment.

COL License Information Items

- COL License Information Item 2.14 Floods

- COL License Information Item 3.5 Flood Elevation

The staff's review of these COL license information items is provided below:

2.4S.4.4.1 Dam Failure Permutations

Information Submitted by Applicant

The applicant considers two permutations for upstream dam failures in the Colorado River Basin. The first permutation considers the simultaneous failure of all dams upstream of Buchanan Dam induced by a seismic event. The recommendation in ANSI/ANS-2.8–1992 (ANSI 1992) is to use a coincidental flood, the lesser of one-half PMF and the 500-year flood, during the failure event. The recommendation is also to use a 2-year wind wave that occurs coincidentally. The applicant states that estimates of the 500-year flood discharges into the Buchanan and Mansfield dams are approximately 10,828 and 14,150 m^3/s (382,400 and 499,700 cfs), respectively. Halff Associates, Inc. (2002) estimated the standard project flood discharges for the two dams as approximately 13,728 and 20,870 m^3/s (484,800 and 737,000 cfs), respectively. The applicant has conservatively selected a coincident flood discharge of 14,158 m^3/s (500,000 cfs) for the two dams.

The second failure permutation considers a domino-type failure of upstream dams with the same coincidental wind and flood events as the first one. However, the applicant assumes the failures to occur in such a way that the combined top-of-dam storage for all dams upstream of Buchanan Dam arrives at the same time before Buchanan Dam fails. The applicant determines that the second of these two permutations would produce the larger flood because the travel and arrival times of the peak discharge are deliberately aligned to produce the largest downstream peak discharge. Therefore, the applicant only analyzes the second permutation for upstream dam breaches in the Colorado River Basin.

FSAR Subsection 2.4S.1.2.2 describes the hydrologic features in the vicinity of STP Units 3 and 4. FSAR Section 2.4S.4 describes flooding due to the postulated domino-type series of dam failures on the Colorado River. The base-case postulated floods coupled with a one-half PMF produces a peak stage of 8.7 m (28.6 ft) MSL. The site is located on the west bank of the Colorado River in Matagorda County, Texas (FSAR Section 2.4S.4). The two large main stem dams are the Buchanan and Mansfield dams, which are at river kilometers 647 and 491 (river miles 402 and 305), respectively, upstream of the site (FSAR Subsection 2.4S.4.1.1). Coupled with a one-half PMF, these dam failures produced a peak stage of 8.7 m (28.6 ft) MSL (FSAR Subsection 2.4S.4.2.1.5) in the base case. The values were lower for a sensitivity case with an increased bottom roughness.

The main cooling reservoir is a manmade reservoir enclosed by a 19.9 km- (12.4 mi)-long embankment. FSAR Subsection 2.4S.1.2.1 discusses the location and function of the main cooling reservoir. The applicant analyzes onsite floods resulting from a postulated instantaneous breach of the north segment of the main cooling reservoir embankment. The main cooling reservoir northern embankment is located about 713 m (2,340 ft) to the south of the STP Units 3 and 4 reactor buildings.

The main cooling reservoir embankment consists of rolled earth approximately 12.2 m (40 ft) high. The interior of the embankment is lined with 0.6 m (2 ft) of thick soil cement, but the outside face is only grass covered. The normal maximum operating water surface elevation in the main cooling reservoir is 14.9 m (49 ft) MSL. The applicant postulates the main cooling reservoir embankment failure mechanisms to include excessive seepage from (1) piping through the foundation of the embankment, (2) seismic activity-induced liquefaction of the foundation of the embankment, and (3) erosion of the embankment from overtopping or from wind-wave events.

The northern portion of the main cooling reservoir embankment is the most critical in terms of a flood wave directed toward the STP Units 3 and 4 site. The applicant considers two breach scenarios, one to the east and the other to the west of the circulating water pipeline.

The applicant uses the HEC-RAS for the river flood routing and RMA2 (Donnell et al, 2008) for routing the flood caused by the postulated main cooling reservoir northern embankment breach. The applicant uses a revision of the Halff study HEC-RAS simulations (Halff Associates, Inc., 2002) for the river flood routing. The applicant uses a bounding calculation to estimate sediment deposition in the STP Units 3 and 4 power block area resulting from the postulated main cooling reservoir northern embankment breach.

The applicant determines the design-basis flood elevation to be 12.2 m (40 ft) MSL, which exceeds the ABWR DCD design value. Therefore, safety-related SSCs will require flood protection. FSAR Section 2.4S.10 describes the flood-protection measures. Because flood levels in the postulated breach of the main cooling reservoir were higher than the site grade, the applicant proposes a departure, STP DEP T1 5.0-1, from the certified ABWR design.

NRC Staff's Technical Evaluation

The staff reviewed the applicant's postulation of dam-failure scenarios on the Colorado River and the main cooling reservoir. The applicant uses two permutations on the Colorado River upstream and one failure scenario on the main cooling reservoir embankment. The applicant also uses the conservative flood events in simulating the Colorado River dam-failure scenarios as recommended by ANSI/ANS-2.8–1992. Based on the applicant's use of ANSI/ANS-2.8–1992, the staff agreed with the applicant's postulations of the dam-failure scenarios and their descriptions. However, the staff found that the applicant did not consider a main cooling reservoir breach scenario caused by a hurricane storm surge in the FSAR. This issue is addressed in Sections 2.4S.5 and 2.4S.10 of this SER. After reviewing the applicant's submittals, the staff determined that a failure of the main cooling reservoir northern embankment would not be caused by a hurricane storm surge.

2.4S.4.4.2 Unsteady Flow Analysis of Potential Dam Failures

Information Submitted by Applicant

The applicant analyzes the upstream dam-failure scenario in the Colorado River Basin using HEC-RAS. The model configuration was based on the earlier study (Halff Associates, Inc., 2002). Several modifications to this earlier modeling effort were motivated by the need to accommodate a more severe flooding event than was previously analyzed in the Halff Associates study (2002). Table 2.4S.4-1 summarizes the configurations of the models before and after the modification.

Table 2.4S.4-1. The Applicant's Modifications to Halff Associates HEC-RAS Model

Model Element	Halff (2002)	FSAR	Rationale
Reach length (km) / (mi)	763 / 474	666 / 414	Applicant routes only downstream of Buchanan Dam
Number of cross sections used	1048	793	Reduced reach length
Bridge crossings	Included	Not included	Assumed to have been washed away
Levees	Included	Some removed as appropriate	Represents more realistic flood propagation
Buchanan reservoir	Baseline Halff (2002)	Modified	Enlarged to accommodate aggregated initial volume of water
Flood plain geometry	Baseline Halff (2002)	Extended using USGS 30-m (98-ft) elevation data set	To allow for larger flow scenario than used in earlier study
Bottom roughness within 4 mi downstream of Buchanan and Mansfield dams[a]	Baseline Halff (2002)	Values used by Halff Associates, Inc. (2002) for Manning's n multiplied by 2	To account for increased roughness due to dam-break debris
Bottom roughness for areas beyond 6 km (4 mi) downstream of failed dams[a]	Baseline Halff (2002)	Values increased by 20% over those used by Halff Associates, Inc. (2002) for Manning's n	To account for increased floodplain roughness due to larger extent than incorporated in earlier study (Halff Associates, Inc. 2002).

(a) The applicant refers to this scenario as the sensitivity case and compares its results with the unmodified base case.

The applicant uses the sum of the maximum water volumes for each of the 56 impoundments upstream of Buchanan Dam as an input for the volume of the stored water in the Buchanan Reservoir to maximize the synchronized peak initial release. The applicant postulates the Mansfield Dam to fail when it overtopped by an estimated overtop depth of 0.9 m (3 ft). The applicant routes the resultant Buchanan and Mansfield dam-break flows with the addition of tributary flow and base-flood flow of 14,158 m^3/s (500,000 cfs) downstream to the river segment adjacent to the STP site.

The applicant analyzes the main cooling reservoir embankment dam failure and the resulting flood hazards using combined simulations of two models: FLDWAV (Fread and Lewis, 1998) and RMA2 (Donnell et al., 2008). The applicant estimates the outflow hydrograph from the main cooling reservoir following a postulated embankment breach using FLDWAV. The applicant then inputs the outflow hydrograph into the RMA2 model to simulate the two-dimensional flood flow outside of the main cooling reservoir embankment. The applicant then performs a bounding calculation to estimate the potential for deposition of these sediments in the STP Units 3 and 4 power block area, in order to determine the potential for an increase in the floodwater surface elevation resulting from the main cooling reservoir northern embankment breach.

The applicant assumes that large concrete structures such as STP Units 1 and 2, STP Units 3 and 4, and other tall and durable structures will remain in place during flooding following the main cooling reservoir embankment breach, while less durable structures, such as metal skin buildings and warehouses, will be mostly removed leaving only the steel framing of these structures in place. The applicant accounts for the effect of these standing structures and other debris by using a higher Manning's n value in the areas where these objects will be present. For the breach-flood modeling, the applicant assumes the bottom elevation of the main cooling reservoir to be between 4.9 and 8.5 m (16 and 28 ft) MSL, with an average bottom elevation of 6.1 m (20 ft) MSL. The applicant assumes the breach side slopes to be 1 horizontal to 1 vertical, and that the breach will expand symmetrically about the center of the breach. As an initial condition of the simulation, the applicant uses a starting main cooling reservoir water surface elevation of 15.5 m (50.9 ft) MSL, which corresponds to a conservative combined effect of a normal maximum operating main cooling reservoir water surface elevation, one-half PMP, and 2-year wind waves.

The applicant uses embankment dam breach parameters recommended for earth-filled structures by the U.S. Bureau of Reclamation (USBR) (Wahl, 1998). The applicant assumes that a service road immediately downstream of the toe of the main cooling reservoir embankment will be eroded away and the terrain further downstream of the road-at approximately 8.8 m (29 ft) MSL—will be the control for the embankment breach bottom elevation. The applicant uses empirical relationships by Wahl (1998) to estimate breach width, time to failure, and peak discharge from the breach. The applicant uses the Froehlich equation to estimate the breach width because it results in the largest estimated breach width. The breach is of a trapezoidal shape, with an average width of 127.1 m (417 ft) and a bottom width of 115.8 m (380 ft). The applicant states that Froehlich's equation results in a conservative estimate of breach width (larger than observed based on a comparison of observed and estimated breach widths of the Teton Dam) and will therefore maximize the discharge through the breach.

The applicant estimates the time to failure of the main cooling reservoir embankment to be 1.7 hours and the peak discharge of 1,772 m^3/s (62,600 cfs) using an equation presented by Wahl (1998). The applicant states that the peak discharge predicted by the FLDWAV model is 3,681 m^3/s (130,000 cfs) 1.7 hours after initiation of the breach, which is more than twice that predicted by Wahl's empirical relationships.

The applicant uses the topography of the STP site, the STP Units 3 and 4 site grading plan, and the STP Units 3 and 4 plot plan to specify the future land surface levels for the RMA2 model. The applicant sets the grade elevation at the center of the power block for STP Units 3 and 4 at 11.2 m (36.6 ft) MSL, and the elevation at the corner of the power block area at 9.8 m (32 ft) MSL. The applicant includes the reactor, turbine, control, radwaste, and service buildings and hot machine shops of all four STP units in the RMA2 model grid. The applicant also includes the UHS for STP Units 3 and 4 and the ECP of STP Units 1 and 2 in the grid. The applicant sets the southern boundary of the RMA2 grid at the northern main cooling reservoir embankment, and extends the grid to FM 521 at its northern end. The applicant selects the east and west boundaries of the RMA2 grid to be far enough away from STP Units 3 and 4 so that conditions at the model grid boundaries will have little influence on simulated variables near STP Units 3 and 4. To ensure model stability, the applicant uses an artificial sump along the eastern, northern, and western boundaries of the RMA2 model grid. The RMA2 grid extends 1,790 m (5,873 ft) in the north-to-south direction and 3,796 m (12,455 ft) in the east-to-west direction. The RMA2 grid includes 2,348 nodes and 1,088 elements varying in size from approximately 232.3 m^2 (2,500 ft^2) near the reactor buildings to approximately 13,378 m^2 (44,000 ft^2) away from STP Units 3 and 4.

The applicant specifies the Manning's n values over the RMA2 grid using published values (Arcement and Schneider, 1989; USACE, 2005). The applicant considers all buildings that are taller than 18.9 m (62 ft) MSL to remain in place during the main cooling reservoir embankment breach flood and these building will totally block the flow. The applicant assigns high roughness values to the area of the buildings that will fail due to the effects of the flood flow.

The downstream boundaries of the model grid are located sufficiently far away so that the maximum flood elevation at the STP Units 3 and 4 safety-related SSCs occurs before the flood front reaches the boundaries. The applicant uses a constant water surface elevation at the downstream boundaries.

The applicant incorporates by reference Section 2.1 of the certified ABWR DCD referenced in Appendix A to 10 CFR Part 52. Because the estimated design-basis flood level is higher than the site grade, the applicant identifies a departure, STP DEP T1 5.0-1, from the certified design. Correspondingly, the applicant proposes flood-protection measures as described in FSAR Section 2.4S.10.

NRC Staff's Technical Evaluation

The applicant uses a combination of FLDWAV and RMA2 to simulate the flood flow following the postulated main cooling reservoir embankment failure and to estimate site characteristics related to this flood. The staff's review of the applicant's analyses considers the following factors:

- whether the applicant uses models that are appropriate for the hydrodynamic problem
- whether the applicant's parameter choices are reasonably conservative
- whether the applicant uses appropriate combinations of events
- whether the applicant correctly selects the design-basis flood and the associated site characteristics
- whether the applicant incorporates an acceptable level of conservatism to provide reasonable assurance for the protection of SSCs important to safety
- whether the applicant's results are reproducible

In RAI 02.04.04-14, the staff asked the applicant to describe how the breach width and breach time parameters were determined and demonstrate that the most conservative plausible breach scenario for the main cooling reservoir embankment was chosen. The applicant responded to the RAI 02.04.04-14 in a letter dated November 22, 2010 (ML110030201).

The staff also determined the need to update the main cooling reservoir embankment breach flood analysis to describe the sensitivity of the flood characteristics to the plausible breach widths and breach time parameters. The staff was not able to determine the characteristics of the design-basis flood at the STP Units 3 and 4 site based on the available information and therefore issued **RAI 02.04.04-13**. This RAI was tracked as **Open Item 2.4.4-1** in the SER with open items. The applicant responded to the RAI 02.04.04-13 as RAI 02.04.04-14 in a letter dated November 22, 2010 (ML110030201).

The staff's review determined that the models and methods used by the applicant in FSAR Section 2.4S.4 are currently used in standard engineering practices. FLDWAV is a generalized flood routing computer program that uses an implicit finite-difference numerical solution scheme to solve the complete one-dimensional St. Venant equations of unsteady flow. FLDWAV Version 2.0.0 was released in June 2000 and has the capability to model time-dependent dam breaches. The staff determined, therefore, that FLDWAV is an appropriate model to use for the initial estimate of flood discharge following the main cooling reservoir embankment breach. However, because FLDWAV is a one-dimensional model, it is not appropriate for use far from the main cooling reservoir embankment, where the flow is expected to spread in two dimensions over a relatively flat terrain.

The staff determined that the applicant has appropriately selected a two-dimensional hydraulic model, RMA2. The applicant has also specified the boundary condition at the southern boundary of the RMA2 computational domain using the results from FLDWAV. RMA2 Waterways Experiment Station (WES) Version 4.5 was released in June 2008 and is a two-dimensional, depth-averaged, finite-element hydrodynamic numerical model that computes water surface elevations and horizontal flow velocities for subcritical, two-dimensional, free-surface flow fields. RMA2 solves the Reynolds form of the Navier-Stokes equations for turbulent flows and has the capability to analyze both steady and unsteady flow problems. The staff determined that RMA2 is an appropriate model to simulate the spreading flood flow following the main cooling reservoir embankment breach.

The staff reviewed the applicant's use of simulation models in estimating the flood following the main cooling reservoir northern embankment breach. The staff's review found that the applicant has appropriately applied the FLDWAV model to simulate the discharge hydrograph resulting from the main cooling reservoir northern embankment breach. The applicant estimates the characteristics of the main cooling reservoir northern embankment breach using a set of empirical approaches. The applicant uses conservatively selected breach characteristics predicted by the empirical approaches as input to the dynamics of breach formation in FLDWAV. The applicant also uses the NWS BREACH (Fread 1991) model to analyze the main cooling reservoir embankment breach and the resulting discharge hydrograph. The applicant uses the predictions from the NWS BREACH model as an independent check of the results from the FLDWAV simulation. The description of the staff's review and confirmatory analysis of the applicant's NWS BREACH application appears below. After reviewing the applicant's method for specifying the bathymetry in the RMA2 model, the staff determined that the applicant has used methods and data sets that are recommended by the FLDWAV and NWS BREACH user manuals.. The staff also reviewed and determined that the variably sized model grid of the RMA2 is appropriate because it uses smaller computational elements near safety-related structures, where the flow is expected to change rapidly. The staff agreed with the applicant's choices for Manning's n because they are reasonably conservative for the expected post-construction conditions in the power block area.

The staff reviewed the combination of events used by the applicant and found that the applicant has followed the dam failure permutation that is recommendations of ANSI/ANS-2.8–1992 (ANS, 1992). Therefore, the staff agreed that the applicant's design-basis flood selection is appropriate for the STP Units 3 and 4 site and RAI 02.04.04-13 is therefore closed.

The staff independently ran the models (NWS BREACH and RMA2) the applicant uses in the main cooling reservoir northern embankment breach flood simulations. The staff also carried out a sensitivity analysis by varying some of the model parameters to determine whether the model results were sensitive to any parameter values. The staff's analyses confirmed that the models produced the same result that the applicant presents in the FSAR. Because the applicant selects model parameters that are recommended for use in current engineering practices, the staff concluded that the applicant's results are reproducible and therefore appropriate for the STP site. The following paragraphs provide details of the staff's independent analyses.

The staff confirmed the applicant's main cooling reservoir northern embankment breach flood discharge and its sensitivity to breach parameters using available and applicable empirical equations and the NWS BREACH model. The staff performed NWS BREACH runs to confirm the applicant's assessment of the main cooling reservoir northern embankment failure flood at the STP Units 3 and 4 site. The staff performed sensitivity studies on the NWS BREACH model parameters.

The staff used data provided by STPNOC in the Units 3 and 4 FSAR responses to various RAIs and in technical reports prepared by STPNOC's contractors. The staff also reviewed other relevant literature (MacDonald and Landridge-Monoplis, 1984; Fread, 1991; and Froehlich, (1995). The NWS BREACH model produces estimates of the breach growth and breach outflow (hydrograph) over time that can be coupled to produce sediment flux over time. The model estimates the growth of the breach based on geometric and hydraulic properties of the embankment and geotechnical parameters of the embankment material. The staff's review determined that the FLDWAV could be used with prescribed timing parameters that specify breach growth, so that the FLDWAV-estimated discharge hydrograph and breach formation approximate those produced by the NWS BREACH (Wahl 2010). If the conceptual model for the subsequent flooding includes multiple or cascading breaches on a river or in a channel network, the FLDWAV would be the appropriate model for simulating the more complex flow scenario. However, in the case of the postulated breach of the main cooling reservoir northern embankment, the conceptual model consists of a single breach with no downstream channel network. Consequently, the staff determined that only the NWS BREACH is necessary to characterize the outflow discharge hydrograph. Therefore, the staff did not use the FLDWAV to estimate the outflow hydrograph from the main cooling reservoir northern embankment breach.

The staff ran the NWS BREACH model using the input file provided by STPNOC. The staff was able to reproduce the results of the NWS BREACH model reported by STPNOC in the applicant's response to RAI 02.04.04-14 (ML110030201). The technical discussion that follows provides the basis for closing RAI 02.04.04-14. The staff used several variations of the NWS BREACH parameters to investigate the sensitivity of model predictions. Based on the NWS BREACH sensitivity analysis, the staff selected a set of conservative parameters that is expected to result in conservative predictions of breach size and peak discharge. The staff varied the elevation at which the piping failure was initiated (Z_p), the length of the dam or main cooling reservoir northern embankment (L), the Manning's roughness parameter (n), the cohesive strength (C), and the friction angle (Ø) of the embankment material.

The staff determined that the NWS BREACH model predictions are fairly insensitive to the elevation at which the piping failure is initiated (Z_p). At a Z_p of 9.1 m (30 ft) MSL and lower, the model run did not finish because of a mathematical error that was probably a result of the Z_p being too close to the bottom of the reservoir embankment. At a Z_p of 14 m (46 ft) and higher, which is very close to the top of the initial water surface elevation in the reservoir, it appears the breach did not develop fully to erode a large portion of the embankment.

The staff determined that the NWS BREACH model predictions are very sensitive to Manning's roughness parameter n. Fread (1991) presents Strickler's equation as

$$n = 0.013\,(D_{50})^{0.67}$$

Where: D_{50} is the median grain size in millimeters (mm).

Using Fread's (1991) version of the Strickler equation gives an estimate of 0.0001 for Manning's n with a D_{50} of 0.001 mm (0.000039 in.). Strickler's equation (USGS, 2011) is presented as:

$$n = 0.015\,(D_{50})^{1/6}$$

The main difference between the Fread (1991) and USGS (2009) equations is the value of the exponent for the median grain size. The USACE (1994) also indicates that the exponent in Strickler's equation should be 1/6 or 0.167:

$$n = 0.034\,(D_{50})^{1/6}$$

The constant in the USACE (1994) equation is 0.034 for natural sediment and D_{50} is in feet.

The difference in the value of the constant could be attributed to units of measurement (USGS 2009). Using the three variations of Strickler's equation with the median grain size of 0.001 mm (0.000039 in.) that the applicant provides, the staff obtained Manning n values of 0.0001 (in the Fread 1991 form); 0.005 (in the USGS 2009 form); and 0.004 (in the USACE 1994 form). For comparison, the recommended lowest Manning n values for smooth brass, Lucite, and glass channels flowing partially full are at least nearly two times greater at 0.009, 0.008, and 0.009, respectively (Chow 1959). Therefore, the staff concluded that Strickler's equation gives unreasonably small estimates of Manning n values for the main cooling reservoir embankment, because the embankment material is expected to form surfaces that would have a much greater roughness than that of metal or glass surfaces. The staff varied the Manning n value from 0.001 to 0.08 to conservatively cover extreme ranges of this parameter. On the basis of NWS BREACH simulation results, the staff determined that the NWS BREACH estimates larger peak flows for larger Manning n values.

The staff also determined that NWS BREACH model predictions are not sensitive to values of C ranging from 1,221 to 1,953 kg/m^2 (250 to 400 lb/ft^2). At values of C lower than 1,221 kg/m^2 (250 lb/ft^2), peak discharge and breach width increased and the time to peak decreased. However, even with a very low cohesive strength of 244 kg/m^2 (50 lb/ft^2), the embankment breach width at peak flow was approximately 156 m (512 ft).

The staff determined that the NWS BREACH model predictions are only slightly sensitive to the frictional angle. Because the east-to-west running portion of the north face of the main cooling reservoir embankment is approximately 1,311 m (4,300 ft) in length, the staff limited the dam length, L, to 1,219 m (4,000 ft). The staff's simulations showed that the NWS BREACH model predictions were not at all sensitive to L values ranging from 304.8 to 1,219 m (1,000 to 4,000 ft). The staff also evaluated the sensitivity of NWS BREACH model predictions to the length of the dam with the cohesive strength set to 488 kg/m^2 (100 lb/ft^2) and the Manning n set to 0.08.

The staff noticed that when the dam length was a limiting factor (L = 152 m [500 ft]), the NWS BREACH model predicted a washout of the entire embankment at the top, but the predicted breach did not grow wider than the length of the embankment itself. When the dam length was not a limiting factor, model predictions were not sensitive to this parameter.

Based on the sensitivity analyses described above, the staff selected a set of fewer parameters to run independent NWS BREACH simulations to conservatively estimate breach size and discharge. Because the NWS BREACH predictions were fairly insensitive to Z_p, C, and Ø, the staff selected the values of these parameters so that they would generally be expected to result in more conservative peak discharge and time to peak parameters. The staff set the initial piping elevation at 9.8 m (32 ft) MSL, the cohesive strength at 976 kg/m^2 (200 lb/ft^2), and the friction angle at 15 degrees. The staff used Manning n values of 0.025, 0.050, and 0.075 in the NWS BREACH simulations listed below:

- Simulation 1: n = 0.025
- Simulation 2: n = 0.050
- Simulation 3: n = 0.075

Simulation 3 yielded the largest peak flow of 3,623 m^3/s (127,929 cfs); the largest breach top and bottom widths of 175.0 and 141.3 m (574.3 and 463.6 ft), respectively; and the shortest time to peak (about 1.99 hr).

The staff also compared the predictions of peak discharge from the NWS BREACH model to historical observations of dam breaches compiled by Wahl (1998). Using the Wahl (1998) database, the staff identified historical breaches of dams that have characteristics similar to those of the main cooling reservoir. The staff used the height of water above the breach, h_w, and the volume stored above the breach bottom, V_w, to compare the embankments listed in Wahl (1998) with the main cooling reservoir, because these two parameters are expected to significantly affect the breach characteristics and subsequent peak discharge. The main cooling reservoir has an h_w of 6.68 m (21.9 ft) and a V_w of 1.88x10^8 m^3 (152,700 acre-ft). The staff searched the historical dam breach database to select entries with an h_w between 4 and 15 m (15 to 50 ft) and a V_w between 1.23 x 10^8 to 3.70 x 10^8 m^3 (100,000 to 300,000 acre-ft) to reasonably bound the corresponding characteristics of the main cooling reservoir. The database contains multiple entries for the same dam failure events if they were reported by several sources. The staff found 172 records that match the selected h_w range in the Wahl (1998) database. These records are associated with 59 unique failure events. Table 2.4S.4-2 shows the breach parameters listed in the database for the 172 records.

Table 2.4S.4-2. Parameters of Historical Dam Breaches With h_w Between 4 and 15 m (15 to 50 ft)

Parameter	Minimum	Maximum
Water height above breach bottom (h_w) (m) [ft]	4.1 [13.3]	15.2 [49.9]
Peak flow (Q_p) (m^3/s) [cfs]	29.4 [1,038]	3,115 [110,005]
Final breach top width	9.2 [30.2]	153.0 [502.0]*

(m) [ft]		
Final breach bottom width (m) [ft]	1.7 [5.6]	97.0 [318.2]
Average final breach width (m) [ft]	4.7 [15.4]	185.9 [609.9]*
Breach formation time (hr)	0.25	1.5
Failure time (hr)	0.5	5.0
*The maximum reported final breach top width in the database is smaller than the maximum reported average final breach width for the 172 selected records. This inconsistency exists in the database because not all breach characteristics are reported for all events.		

The database lists nine entries that include volumes above the breach bottom, V_w, in the ranges of interest. Seven of these records are associated with the Teton Dam failure, and the remaining two are associated with the Martin Cooling Pond failure. Table 2.4S.4-3 lists the parameters for the Teton Dam and Martin Cooling Pond failures.

Table 2.4S.4-3. Parameters of Historical Dam Breaches With V_w Between 1.23×10^8 to 3.70×10^8 m^3 (100,000 to 300,000 acre-ft)

Parameter	Teton Dam	Martin Cooling Pond
Water height above breach bottom (h_w) (m) [ft]	67.1–83.8 [219.9–275.0]	8.5 [28]
Volume of water above breach bottom (V_w) (m^3) [acre-ft]	3.10×10^8 [251,321]	1.36×10^8 [110,257]
Peak flow (Q_p) (m^3/s) [cfs]	65,120–65,136 [2,299,691–2,300,256]	3,115 [110,005]
Final breach top width (m) [ft]	Not available	Not available
Final breach bottom width (m) [ft]	Not available	Not available
Average final breach width (m) [ft]	151 [495]	185 [607]
Breach formation time (hr)	1.25	Not available

Failure time (hr)	4	Not available

The only entries in the database that are similar in conditions with the STP main cooling reservoir are those associated with the case of the Martin Cooling Pond embankment failure. The NWS-BREACH predictions of for the main cooling reservoir are similar to those observed for the Martin Cooling Pond. The Martin Cooling Pond has a larger h_w of 8.5 m (28 ft) compared to the main cooling reservoir value of 6.7 m (21.9 ft). The Martin Cooling Pond has a smaller V_w of 1.36 x 10^8 m^3 (110,257 acre-ft) compared to the main cooling reservoir value of 1.88 x 10^8 m^3 (152,700 acre-ft). The final average breach width for Martin Cooling Pond was 185 m (607 ft) compared to the NWS BREACH-predicted main cooling reservoir average breach width at peak flow of 210 m (688 ft). On the basis of this comparison, the staff concluded that the NWS-BREACH predictions are reasonable and conservative for the postulated main cooling reservoir northern embankment failure.

The staff determined that Simulation 3 is the most conservative of the NWS BREACH simulations. Therefore, the staff used the discharge hydrograph from this simulation as input to the RMA2 model.

The staff conducted a series of RMA2 confirmatory and sensitivity analyses to evaluate the flooding at the STP Units 3 and 4 site due to a breach of the main cooling reservoir northern embankment. The postulated breach location was about 762 m (2,500 ft) away from the site. The staff's sensitivity analyses were based on the RMA2 hydrodynamic model setup provided by the applicant.

The applicant uses two postulated main cooling reservoir northern embankment breach scenarios. These two scenarios use the same breach widths (140 m [460 ft]) and peak discharge (3,653 m^3/s [129,000 cfs]), but the scenarios vary in the location of the breach on the main cooling reservoir northern embankment. They are both called east and west embankment breach scenarios.

The staff confirmed that the applicant's hydrodynamic model setup (boundary conditions) is consistent with recommendations in the literature or in the RMA2 User's Manual. The staff also determined that the applicant's values for parameters (such as Manning's roughness coefficient and turbulent exchange coefficients) for the post-construction conditions expected in the power block area are reasonably conservative and are based on values reported in the literature and in the RMA2 User's Manual.

The applicant uses an artificial sump near the open boundary of the RMA2 simulation domain to avoid model instability. The applicant states that the sensitivity analysis performed for the fixed elevation boundary condition in the artificial sump does not affect the floodwater surface elevation at the STP Units 3 and 4 site. The staff agreed that the effect of the artificial sump would not be significant, because the artificial sump is located relatively far from the area of interest where STP Units 3 and 4 safety-related SSCs would be located.

The applicant sets the open downstream boundary condition at 9.9 m (32.5 ft) MSL. The applicant also describes a sensitivity analysis in FSAR, Section 2.4S.4.2.2.4.1, which examines the effect of increasing the open downstream boundary condition to an elevation of 10.4 m (34 ft) MSL. The applicant states that the effect on the floodwater surface elevation at the STP site because of a change in the open downstream water surface elevation is minor. The staff used the discharge hydrograph generated by the NWS BREACH model (Simulation 3 above) to specify the upstream boundary condition to the RMA2 model. The staff used two scenarios for the RMA2 simulations. The first scenario consisted of the discharge hydrograph obtained from NWS BREACH Simulation 3, with the downstream open boundary condition in the RMA2 grid set to 9.9 m (32.5 ft) MSL. In the second scenario, the staff used the same discharge hydrograph at the upstream boundary in the RMA2 grid, but changed the downstream open boundary condition to 11.0 m (36 ft) MSL to determine whether the choice of the downstream open boundary condition setting significantly affects the floodwater surface elevation at the STP Units 3 and 4 site.

The RMA2 model needs a "spin-up" before applying the breach discharge hydrograph as a boundary condition. A dynamically consistent combination of water surface elevations and flow patterns is necessary as an initial condition for RMA2; a flat water surface and no flow with water over the entire model domain is one such condition. However, before the discharge resulting from the main cooling reservoir embankment breach arrives at the upstream boundary of the RMA2 model domain, the modeled area will be dry. To reconcile the model requirement with reality, the staff initially set the water surface elevation at 20.1 m (66 ft) MSL with a small discharge. The staff then linearly decreased the water surface elevation at the downstream open boundary to an elevation equal to that used as the final open boundary condition (9.91 m [32.5 ft] MSL for the first scenario and 11.0 m [36.0 ft] MSL for the second scenario). After the "spin-up" period, the RMA2 model domain would have a small water depth with a small discharge. The staff then applied the NWS BREACH Simulation 3 hydrograph at the upstream boundary while keeping the water surface elevation constant at the downstream open boundary. The staff performed two RMA2 simulations for the east and the west breach scenarios used by the applicant.

Table 2.4S.4-4 includes the staff's summary of the predicted water surface elevations in the RMA2 simulations at the same locations as those shown in FSAR Figure 2.4S.4-19, except for Location 7, which is near the breach and is not in the power block area. The staff noted that increasing the specified water surface elevation at the downstream open boundary results in slightly higher water surface elevations in the power block area. This increase is about 0.08 m (0.25 ft). Because the increase in water surface elevation is small and the specified elevation is conservatively chosen, the staff concluded that the effect of the chosen downstream open boundary condition is minor. The staff conducted independent RMA2 simulations and concluded that the estimated maximum water surface elevation in the power block area would be 11.9 m (39.04 ft) MSL.

Table 2.4S.4-4. East Breach Peak Flood Elevations (m [ft] MSL)

Scenarios	Locations						
	Unit 4 North	Unit 3 North	Unit 4 South	Unit 3 South	Unit 4 UHS South	Unit 3 UHS South	Between Units 3 and 4
East Breach Scenario 1 (BREACH Simulation 3)	11.00 (36.09)	10.90 (35.76)	11.55 (37.90)	11.66 (38.26)	11.70 (38.39)	**11.79 (38.69)**	11.45 (37.56)
East Breach Scenario 2 (BREACH Simulation 3 with open downstream	11.16 (36.62)	11.13 (36.53)	11.62 (38.13)	11.74 (38.51)	11.77 (38.63)	**11.87 (38.94)**	11.50 (37.73)

boundary set to 11.0 m [36.0 ft] MSL)							
West Breach Scenario 1 (BREACH Simulation 3)	11.04 (36.22)	10.87 (35.66)	11.63 (38.16)	11.68 (38.31)	**11.82 (38.79)**	11.60 (38.05)	11.45 (37.55)
West Breach Scenario 2 (BREACH Simulation 3 with open downstream boundary set to 11.0 m [36.0 ft] MSL)	11.18 (36.69)	11.11 (36.45)	11.70 (38.40)	11.76 (38.58)	**11.90 (39.04)**	11.70 (38.38)	11.50 (37.73)
Values in boldface indicate the maximum floodwater surface elevation for each scenario.							

Because the discharge following the postulated breach of the main cooling reservoir northern embankment is expected to carry a large amount of eroded embankment material, a significant deposition of this sediment could occur at the STP Units 3 and 4 site. The staff performed a bounding calculation to conservatively estimate a potential change in the topography of the power block area of STP Units 3 and 4 resulting from the postulated northern main cooling reservoir embankment breach. The flood would carry scoured embankment sediments and sediment from a scour hole postulated to form adjacent to the location of the breach. The staff conservatively assumed that all of the combined mobilized sediment would deposit in the power block area, therefore resulting in an additive upward shift of the maximum floodwater surface elevation estimated by the RMA2 model.

The staff used descriptions of embankment geometry (Bechtel Energy Corporation, 1984) and the NWS BREACH model to estimate the breach width. The staff computed the volume of eroded embankment material using the NWS BREACH-predicted final average breach width of 209.7 m (687.9 ft). The staff estimated the volume of eroded embankment sediment to be 88,103 m^3 (3,111,318 ft^3). The staff doubled the applicant's estimate of the scour hole volume of 43,693 m^3 (1,543,000 ft^3) to conservatively account for uncertainty in the dimensions of the postulated scour hole. Therefore, the staff's estimate of total volume of mobilized sediment is 175,489 m^3 (6,197,318 ft^3).

The staff postulated that in the bounding case, all of the mobilized sediment could be deposited in the power block area. The staff used FSAR Figure 2.4S.4-15 to estimate the site area where all sediment is postulated to deposit. The staff's estimated size of this area is approximately 924.8 m (3,034 ft) by 882.7 m (2,896 ft). The staff also estimated that 20 percent of this area could be covered by buildings and consequently, would not be available for deposition. Therefore, the staff estimated the area available for deposition to be 653,031 m^2 (7,029,171 ft^2). The staff also estimated that evenly distributing the total mobilized sediment volume over this area would yield a uniform thickness of 0.27 m (0.88 ft). Because the water velocity during the flood would be significant, the staff determined that a significant portion of the mobilized sediment would likely be carried beyond this area. Therefore, the assumption that all of the mobilized sediment would deposit in the selected area is conservative.

To conservatively estimate the maximum water surface elevation under the bounding sediment deposition scenario, the staff added the bounding estimate of uniform sediment deposition thickness to the maximum water surface elevation estimated in the power block area resulting from the postulated northern main cooling reservoir embankment breach. Consequently, the staff estimated the maximum floodwater surface elevation under the bounding sediment deposition scenario in the power block area to be 12.2 m (39.9 ft) MSL. Therefore, the staff determined that sediment deposition in the power block area of STP Units 3 and 4 would not result in a floodwater surface elevation that exceeds 12.2 m (39.9 ft) MSL.

2.4S.4.4.3 Water Level at the Plant Site

Information Submitted by Applicant

The highest water surface elevation during the RMA2 simulations, 11.8 m (38.8 ft) MSL, occurred at the STP Unit 4 UHS structure for the west breach scenario, approximately 1.75 hours after the breach. The peak flow velocity of approximately 1.44 m/s (4.72 fps) occurred between STP Units 3 and 4 approximately 1.75 hours after the breach. The applicant performs a sensitivity analysis by changing the downstream boundary condition from a constant elevation of 9.9 m (32.5 ft) MSL to 10.4 m (34 ft) MSL. This change does not affect the peak floodwater surface elevations at the STP Units 3 and 4 site.

The applicant selects the design-basis floodwater surface elevation of 12.2 m (40 ft) MSL at the STP Units 3 and 4 site.

Sedimentation and Erosion

The applicant estimates that the main cooling reservoir embankment will contribute approximately 48,138 m^3 (1.7 million ft^3) of clay, 2,142 m^3 (75,644 ft^3) of sand, and 3,329 m^3 (117,562 ft^3) of soil cement to the flood. The applicant also estimates that the flood following the main cooling reservoir embankment breach will produce a scour hole approximately 6.1 m (20 ft) deep, 61.9 m (203 ft) long, and 115.8 m (380 ft) wide and will therefore contribute approximately 42,475 m^3 (1.5 million ft^3) of clay to the flood flow.

The applicant estimates that the flood following the main cooling reservoir embankment breach will not cause severe erosion of concrete, asphalt, compacted gravel, and grass surfaces near the plant area of STP Units 3 and 4. Some minor erosion around the corners of buildings will be expected, but the applicant expects that the safety-related functions will not be adversely affected by this minor erosion.

In a revised response to RAI 02.04.04-15 dated November 22, 2010 (ML111150106), the applicant describes a bounding analysis of sediment accumulation in the STP Units 3 and 4 power block area resulting from the postulated main cooling reservoir northern embankment breach. The applicant uses a sediment volume estimate of 9,756 m^3 (3,433,517 ft^3) that includes contributions from the main cooling reservoir embankment and from the formation of a scour hole adjacent to the postulated breach site. The applicant doubles this sediment volume for conservatism. The applicant also assumes that the entire volume of sediment deposits evenly near the STP Units 3 and 4 plant area. The applicant identifies the dominant flow path developed within the RMA2 simulations and selects a fan-shaped area extending northward from the postulated breach location to the FM 521 Road. The applicant uses the RMA2 computation mesh covering the fan-shaped area to estimate its size and excludes areas covered by buildings. The applicant's estimates of deposition areas are 1,825,227 m^2 (19,646,580 ft^2) and 1,667,482 m^2 (17,948,623 ft^2) for the east and west breach scenarios, respectively. The applicant computes the deposition thickness for the east and west breach scenarios by dividing the sediment volume by the respective deposition areas. The applicant's estimates of sediment deposition thicknesses are 0.11 m (0.35 ft) and m 0.12 m (0.38 ft), respectively. The applicant rounds the deposition thickness upward to 0.12 m (0.40 ft). The applicant conservatively assumes that a maximum floodwater surface elevation resulting from the main cooling reservoir northern embankment breach would be raised by the deposition thickness estimate. The applicant's estimate of a maximum floodwater surface elevation resulting from the postulated breach accounting for a potential sediment deposition in the STP Units 3 and 4 power block area is 11.9 m (39.2 ft). The applicant states that this revised maximum floodwater surface elevation is below the design basin flood elevation of 12.2 m (40 ft) for STP Units 3 and 4.

Hydrodynamic Forces

The staff reviewed the applicant's estimation of sedimentation and erosion using the SED-2D model. The staff determined that the applicant did not provide sufficient information on SED – 2D modeling. Therefore the staff issued RAI 02.04.04-15. In response to RAI 02.04.04-15, dated March 28, 2011 (ML110890901) the applicant estimates the hydrodynamic loading on plant buildings from maximum floodwater surface elevations and flow velocities. For the east and west breach scenarios, the applicant reports maximum flow velocities of 1.44 and 1.43 m/s (4.72 and 4.68 fps), respectively, at seven selected locations in the power block area (FSAR Figure 2.4S.4-19). The applicant estimates the suspended sediment concentration during the peak discharge to be 22.33 kg/m^3 (1.394 lb/ft^3). The applicant rounds the suspended sediment concentration upward to 23 kg/m^3 (1.44 lb/ft^3) and computes a sediment-laden fluid density of 1,023 kg/m^3 (63.86 lb/ft^3). The applicant uses a conservative maximum sediment concentration of 23 kg/m^3 (1.44 lb/ft^3), along with the maximum flow velocity, to estimate the drag force on plant buildings as approximately 214.8 kg/m^2 (44 lb/ft^2).

Spatial Extent of Flooding Due to Main Cooling Reservoir Embankment Breach

Using the topographic features near the STP site, the applicant estimates that most of the floodwaters released following the main cooling reservoir embankment breach will spread out over the area bounded by FM 521. The approximate top elevation of FM 521 ranges between 8.5 and 9.1 m (28 to 30 ft) MSL. North of FM 521 and west of the main cooling reservoir, there are levees with top elevations of approximately 7.6 to 9.1 m (25 to 30 ft) MSL. The general slope near the STP site is toward the Colorado River to the east. Therefore, the applicant concludes that after the main cooling reservoir embankment breach, most of the floodwater will flow east toward the river. However, a portion of the flow will likely reach the LRS, then flow south along the west main cooling reservoir embankment, and eventually reach the Gulf Intracoastal Waterway. The applicant concludes that it is unlikely that the flood will overtop FM 521 and the levees located west of the STP site. However, if this were to happen, some flow could also reach the Tres Palacios River located west of the STP site.

Water Level at the STP Units 3 and 4 Site from Failure of Upstream Dams

Using the HEC-RAS simulation, the applicant estimates the maximum water surface elevation during the upstream dam breach as 8.7 m (28.6 ft) MSL. The applicant estimates the coincident wind waves for the upstream dam-failure scenario at the STP Units 3 and 4 site using the 2-year wind according to the methods described in the Coastal Engineering Manual (USACE 2008). The applicant reports that an accurate estimate of fetch length for this flood scenario cannot be made, which is also documented in the STP Units 1 and 2 UFSAR. Based on topographic variations and manmade features that may limit wind effects, the applicant identifies two critical fetches: one toward the east and the other toward the northeast of the STP Units 3 and 4 site.

The applicant estimates the fetch toward the east to be approximately 24.9 km (15.5 mi) long, with the maximum water depth along the fetch varying from 0.3 to 7 m (1 to 23 ft) during the peak discharge. The applicant estimates the northeast fetch to be approximately 28.3 km (17.6 mi) long, with the maximum water depth along the fetch varying from 0.3 to 2.7 m (1 to 9 ft) during the peak discharge. The applicant estimates the maximum wind setup to be approximately 1.2 m (3.9 ft). Based on the available input estimates and data, the applicant estimates the combined water surface elevation near the STP Units 3 and 4 power block area to be approximately 9.9 m (32.5 ft) MSL, with a water depth of approximately 1.4 m (4.5 ft) because the surrounding site grade around the power block and UHS is nominally 8.5 m (28 ft) MSL. The applicant concluded that because of the shallow water depth, breaking wave conditions would occur and the estimated breaking wave height would be 1.1 m (3.5 ft).

The outward slope of the power block area will be at 10 horizontal to 1 vertical. The applicant estimates the maximum wave runup to be 0.6 m (1.9 ft). Therefore, the applicant estimates that the maximum water surface elevation near the STP Units 3 and 4 power block area to be 10.5 m (34.4 ft) MSL.

The applicant incorporates by reference Section 2.1 of the certified ABWR DCD referenced in Appendix A to 10 CFR Part 52. Because flood levels from the postulated breach of the main cooling reservoir were higher than the site grade, the applicant identifies a departure, STP DEP T1 5.0-1, from the certified design. Flood protection will be needed for safety-related SSCs of STP Units 3 and 4, as described in FSAR Section 2.4S.10.

NRC Staff's Technical Evaluation

The applicant's FSAR did not clearly describe the spatial extent of flooding during the postulated main cooling reservoir breach. Therefore, the staff issued RAI 02.04.04-9 requesting the applicant to evaluate the spatial extent of flooding during the postulated main cooling reservoir breach and to evaluate whether the flood from the postulated main cooling reservoir breach will cause an overflow of any basin ridgelines. The applicant responded to RAI 02.04.04-9 in letters dated January 28 and February 23 of 2009 (ML090300648 and ML090710301).[1] The applicant states in the FSAR that a small portion of the flow following the postulated failure of the main cooling reservoir embankment could overflow into the Tres Palacios River if the flood were to overtop the levees located toward the west of the STP site. The applicant states, however, that most of the flow will eventually flow to the east to the Colorado River or to the south via the LRS to the Gulf Intracoastal Waterway.

The staff reviewed the applicant's response and the main cooling reservoir embankment breach flood simulation, and determined that it is unlikely that a large portion of the main cooling reservoir water would cross the basin ridgelines. The depth of flow at the STP Units 3 and 4 site is approximately 1.8 m (6 ft) and gets progressively smaller at distances farther from the main cooling reservoir embankment breach. The RMA2 simulations show that flow toward the west starts to be intercepted by the LRS and begins to turn southward. The velocity of flow in this region is less than 0.6 m/s (2 fps). Therefore, the staff concluded that it is unlikely that the flow would overtop the levees located to the west of the STP site and RAI 02.04.04-9 is considered closed.

[1] The attachments to the letter dated February 23, 2009, which contain the applicant's RAI response, are in ADAMS Accession Numbers ML090710302 and ML090710304.

The staff issued RAI 02.04.04-10 requesting the applicant (1) to discuss the composition of the flood wave (essentially a mudflow) with respect to the sediment generated from the postulated breach of the main cooling reservoir embankment and carried with the flow, including dynamic and impact forces, and to discuss the conservatism of this case compared to the case presented in the FSAR; and (2) to discuss the effects of the settlement of bank materials resulting from the postulated failure of the main cooling reservoir embankment that could result in an accumulation of a large amount of bank material at the plant site, specifically, the effects on the safety-related structures and the operation of the plant after the postulated main cooling reservoir failure and to explain how these effects, if significant, will be addressed in Section 2.4S.14, "Technical Specifications and Emergency Operations Requirements." The applicant responded to RAI 02.04.04-10 in letters dated January 28 and February 23 of 2009 (ML090300648 and ML090710301). Subsection 2.4S.4.4.3, titled "Information Provided by the Applicant," of this SER includes a summary of the applicant's bounding calculation for sediment deposition in the STP Units 3 and 4 power block area.

To estimate sediment concentrations associated with the peak flow conditions in BREACH simulations, the staff examined the change in the breach geometry and in the volume of eroded embankment material during the short period when the discharge is near its maximum. The staff converted the sediment volume to a sediment mass to estimate the sediment concentration at peak discharge. The staff estimated the sediment concentration to be 2.6 kg/m^3 (0.16 lb/ft^3), which is attributable to the contribution from the embankment but does not include contributions from the scour hole. The staff assumed that the embankment material would be dense (2,650 kg/m^3 [165 lb/ft^3]) and fully compacted (porosity = 0) to make this estimate conservative.

The staff used the applicant's estimate for the scour hole dimensions and then doubled it to account for uncertainty. The staff assumed that the scour hole was completely formed at the time of peak flow. The staff assumed a linear rise in flow to its peak value in order to compute total water volume discharged during formation of the scour hole. The staff computed the average sediment concentration during scour hole formation as the total scoured sediment volume divided by the discharged water volume. The staff's assumption of a dense scour hole material and full compaction is conservative with respect to the calculation of the scour hole contribution to the sediment concentration at peak discharge. The staff's estimate of the scour hole contribution to sediment concentration is 20.1 kg/m^3 (1.25 lb/ft^3). The staff's combined sediment concentration estimate is therefore 22.7 kg/m^3 (1.42 lb/ft^3). The staff assumed that the sediment concentration remains unchanged between the locations of the breach and power block area. The staff considered this assumption to be conservative because most of the suspended sediment would be derived from the embankment and scour hole, and because the staff doubled the applicant's estimate of the volume of sediment derived from the scour hole.

The staff conservatively estimated the density of the sediment-laden floodwater by adding the water density to the sediment concentration to obtain a combined density of 1,022.7 kg/m^3 (63.8 lb/ft^3) or an increase of 2.3 percent more than the density of water with no sediment. The staff determined that because the drag is linearly related to fluid density, it would increase 2.3 percent more than that caused by water with no sediment.

The staff examined to the RMA2 results for the main cooling reservoir west embankment breach scenario. In addition to the seven locations examined by the applicant, the staff examined maps of the velocity magnitude in the power block area and found that the maximum velocity magnitude was about 2.10 m/s (6.9 fps) when the downstream boundary condition was held at 9.9 m (32.5 ft) MSL and about 2.13 m/s (7.0 fps) when held at 11.0 m (36.0 ft) MSL. The staff found that in the RMA2 results, the velocity magnitudes were generally lower when the downstream boundary was held at a higher value. However, in some localized areas, the velocities were slightly higher.

The drag force, F, on the building wall is computed at the product of a drag coefficient, C_d, the fluid density, ρ, and the squared fluid speed, V, divided by twice the acceleration due to gravity, g:

$$F = C_d \rho V^2 / (2 g)$$

A conservative value of C_d is 2.0, freshwater has a density of 1000 kg/m^3, and g is 9.81 m/s^2 (3.2.2 ft/s^2). The staff estimated that suspended sediment from the embankment breach is 2.6 kg/m^3 (0.16 lb/ft^3) and the contribution from the scour hole is 20.1 kg/m^3 (1.25 lb/ft^3). Therefore, the fluid density, ρ, is conservatively estimated as 1022.7 kg/m^3 (63.845 lb/ft3) or 2.3 percent larger than that of freshwater. The staff examined the RMA2 results and found that the maximum velocity in the power block area, V, in the RMA2 simulation is 2.13 m/s (7.0 ft/s). Using the above equation, a conservative value for the drag coefficient, a combined water and sediment fluid density, and a maximum water velocity, the staff computed that the maximum drag force on the power block buildings due to flooding caused by the postulated main cooling reservoir breach and subsequent flood is 485 N/m^2 (99.3 lb/ft^2).

The staff issued RAI 02.04.04-11 stating, "In response to RAI 02.04.04-9 and 02.04.04-10 (U7-C-STP-NRC-090012, February 23, 2009; Attachment 1), the applicant proposed changes to the FSAR. The proposed text for FSAR Subsection 2.4S.4.2.2.3.1 mentions that a hypothetical sump was modeled at East, West, and North boundaries. Is this configuration simply a deepening of the topography along these boundaries when the water surface elevation is held constant? How were the sumps added to the model and how were they incorporated with the specified boundary conditions? RMA2 model description suggests that these sumps were needed to improve model stability. What is the nature of the instability that is being addressed? Provide citations to publicly available references that describe this approach while using the RMA2 model."

In a letter dated August 26, 2009 (ML092430134), the applicant's response to RAI 02.04.04-11 states that a common reason for numerical instability in dynamic models is the oscillation of boundary nodes between wet and dry conditions. The applicant provides a set of references that use such an approach. The applicant also states that an artificial sump is used with the topographic elevations of nodes within the sump set to a low value so that they always remain wet. Most modeling guides recommend that the boundaries should be located far away from the region of interest, because the effects of the selected conditions at remote boundaries are less likely to affect predicted variables such as the water surface elevation in the area of interest. The applicant notes that the RMA2 model setup for the STP Units 3 and 4 site has experienced instability problems, including nonconvergence and early termination of the simulation. The applicant uses an artificial sump along the boundaries to ensure the removal of the instability. The applicant also performs a sensitivity analysis to verify that the water surface elevations set for the artificial sump will not significantly change the predicted hydraulic conditions near the STP Units 3 and 4 power block area.

The staff reviewed the applicant's response, including the references the applicant provided. On the basis of this review, the staff determined that the applicant's use of artificial sumps in RMA2 modeling is appropriate. The staff also reviewed the applicant's sensitivity analysis and agreed that the use of artificial sumps will not significantly change the flow characteristics near the STP Units 3 and 4 power block.

The staff issued RAI 02.04.04-12 stating, "In response to RAI 02.04.04-9 and 02.04.04-10 (U7-C-STP-NRC-090012, February 23, 2009; Attachment 1), the applicant proposed changes to the FSAR. The proposed text for FSAR Subsection 2.4S.4.2.2.3.2 discussed the impact of treating buildings in the main cooling reservoir breach analysis as 'hard' or 'soft.' The response states that considering the buildings as 'soft' results in a conservative estimate of flood inundation. It is not clear if this is general statement or finding from this particular model analysis. The conclusion made in the RAI response (applicant's response to RAI 02.04.04-3, in U7-C-STP-NRC-090022, Attachment 4, page 1 of 4) is not clear to the staff because removal of obstructions ('soft' buildings) may increase the cross-sectional area of the discharge even though the roughness in those areas may have been increased. Provide a discussion on why removal of 'soft' buildings would result in higher floodwater surface elevations and greater velocities."

In a letter dated August 26, 2009 (ML092430134), the applicant's response to RAI 02.04.04-12 states that the classification of buildings as "hard" or "soft" is based on an engineering judgment. The applicant also states that the removal of "soft" buildings located directly between the main cooling reservoir embankment breach and the STP Units 3 and 4 power block will also remove obstructions to flood flow and therefore, will cause a greater flood inundation. The applicant notes that the removal of "soft" buildings will make the results of the analysis more realistic.

The staff reviewed the applicant's response and determined that the removal of "soft" buildings, where the flow velocities following the main cooling reservoir embankment breach are high, would make the flooding at the STP Units 3 and 4 power block area more realistic, because some of the structures in the area were not designed to withstand the flood event caused by a postulated main cooling reservoir northern embankment breach. The staff also determined that the cross-sectional area of the removed obstructions will be relatively small compared with the cross-sectional area of the flood flow. Therefore, the staff concluded that any increase in the cross-sectional area of flow because of the removal of soft buildings would likely be minor, and the corresponding decrease in the flow velocity would also be minor. Consequently, the staff concluded that the net change in the design-basis floodwater surface elevation would also be minor.

The applicant provided a bounding calculation dated March 28, 2011 (ML110890901) to address the effects of sediment deposition at the STP Units 3 and 4 site. In addition, the staff determined that the flood induced by the postulated failure of the northern main cooling reservoir embankment has the potential to cause erosion at the site. Because the staff did not have detailed information regarding geotechnical and hydrologic properties of the post-construction top surface within and near the power block area, the staff was unable to estimate the characteristics of site-specific erosion during the flood. Therefore, the staff adopted a bounding approach and conservatively determined that the clay layer provided above the backfill material within the power block area could be eroded away. The staff postulated that infiltration of the floodwater could occur when the clay layer is eroded and the backfill material is exposed to floodwaters. Section 2.4S.12 of this SER provides an assessment of the effects of the postulated infiltration.

As stated above in Section 2.4S.4.4.2 of this SER, the staff determined that the maximum floodwater surface elevation at the STP Units 3 and 4 site during the main cooling reservoir embankment breach event would not exceed 12.2 m (39.9 ft) MSL. Therefore, the staff determined that the applicant's design-basis flood elevation of 12.2 m (40 ft) MSL is acceptable. Therefore RAIs 02.04.04-10, 02.04.04-11, 02.04.04-12 and 02.04.04-15 are considered closed. In response to RAI 02.04.04-14 and 02.04.04-15, the applicant proposed to revise the FSAR Section 02.04S.04 to clarify the process of main cooling reservoir embankment breach modeling and the effects of erosion and sedimentation on the design basis flood level and the maximum groundwater level. Revisions to the FSAR for RAI 02.04.04-14 and 02.04.04-15 will be tracked as Confirmatory Item 02.04.04-1.

2.4S.4.5 Post-Combined License Activities

There are no post-COL activities related to this subsection.

2.4S.4.6 Conclusion

The staff reviewed the application and confirmed that the applicant has addressed the required information related to estimates of the flood characteristics caused by the postulated dam break scenarios, including the main cooling reservoir embankment breach. The staff conducted independent analyses and confirmed that the applicant's design-basis floodwater surface elevation of 12.2 ft (40 ft) MSL is acceptable. The staff also reviewed the applicant's bounding calculations used to estimate the sediment deposition in the power block area as a result of the main cooling reservoir embankment breach and how this low conductivity waste deposition would affect floodwater surface elevations in the power block area. The staff's independent estimate of the additional increase in the floodwater surface elevation under a bounding sediment deposition scenario confirmed that the floodwater surface elevation in the power block area of STP Units 3 and 4 would not exceed 12.2 m (40 ft) MSL. The staff concluded that the surface water elevations expected during the postulated northern main cooling reservoir embankment breach event is the design-basis flood for the safety-related SSCs at the STP Units 3 and 4 site.

2.4S.4.7 References

Arcement, G.J., and V.R. Schneider, "Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains," U.S. Geological Survey, Water Supply Paper 2239, 1989.

Bechtel Energy Corporation, "Evaluation of Strength Parameters and Stability Main Cooling Reservoir Embankment. Volume 1 Report," Harza Engineering Company, Houston, Texas, 1984.

Donnell, B.P, J.V. Letter, W.H. McAnally, and W.A. Thomas, "User's Guide for RMA Version 4.5," 2008.

Fread, D.L., "BREACH: An Erosion Model for Earthen Dam Failures," Hydrologic Research Laboratory, National Weather Service, NOAA, Silver Spring, MD, Revision 1, 1991.

Fread, D.L., J.M. Lewis, "NWS FLDWAV Model: Theoretical Description and User Documentation," Hydrologic Research Laboratory Office of Hydrology, National Weather Serivce, NOAA, Silver Spring, MD, 1998.

Halff Associates, Inc., "1992 Colorado River Flood Damage Evaluation Project, Final Report, Phase I, Volume I and Volume II," prepared for the Lower Colorado River Authority and Fort Worth District Corps of Engineers, July 2002.

MacDonald, T.C. and J. Langridge-Monopolis, "Breaching Characteristics of Dam Failures," *Journal of Hydraulic Engineering*, Volume 110, Number 5, Paper No. 18795.

South Texas Nuclear Operating Company, "South Texas Project Combined License Application," Revision 0, Part 2, Final Safety Analysis Report, 2007.

South Texas Nuclear Operating Company 2010, Letter from Scott Head to NRC Document Control Desk, U7-C-STP-NRC-100241, November 22, 2010.

U.S. Army Corps of Engineers, "Hydraulic Design of Flood Control Channels," Engineer Manual EM 1110-2-1601, Washington DC, 1994.

U.S. Army Corps of Engineers, "HEC-RAS, River Analysis System, User's Manual," Version 3.1.3, Hydrologic Engineering Center, 2005, U.S. Geological Survey, "Water Resources of Illinois: n-Values Project," available at http://il.water.usgs.gov/proj/nvalues/equations.shtml?equation=05-strickler, accessed March 15, 2011, last updated February 28, 2009.

Wahl, T.L., "Prediction of Embankment Dam Breach Parameters, A Literature Review and Needs Assessment," Dam Safety Research Report DSO-98-004, Dam Safety Office, Water Resources Research Laboratory, U.S. Department of the Interior, Bureau of Reclamation, 1998.

Wahl, T.L., "Dam Breach Modeling—An Overview of Analysis Methods," Joint Federal Interagency Conference on Sedimentation and Hydrologic Modeling, June 27–July 1, 2010, Las Vegas, Nevada, 2010.

Wahl, T.L., "Prediction of Embankment Dam Breach Parameters – A Literature Review and Needs Assessment," Water Resources Research Laboratory, U.S. Department of the Interior Bureau Of Reclamation Dam Safety Office, Dam Safety Research Report DSO-98-004, July 1998.

2.4S.5 Probable Maximum Surge and Seiche Flooding

2.4S.5.1 Introduction

This section of the FSAR addresses the probable maximum surge and seiche flooding to ensure that any potential hazard to the safety-related SSCs at the proposed site has been considered in compliance with the Commission regulations.

This SER section presents the evaluation of the following topics based on data provided by the applicant in the FSAR and information available from other sources: (1) probable maximum hurricane (PMH) that causes the probable maximum surge as it approaches the site along a critical path at an optimum rate of movement; (2) probable maximum wind storm (PMWS) from a hypothetical extratropical cyclone or a moving squall line that approaches the site along a critical path at an optimum rate of movement; (3) a seiche near the site and the potential for seiche wave oscillations at the natural periodicity of a water body that may affect the elevations of the flood-water surface near the site or cause a low water surface elevation affecting safety-related water supplies; (4) wind-induced wave runup under PMH or PMWS winds; (5) effects of sediment erosion and deposition during a storm surge and seiche-induced waves that may result in blockage or loss of function of SSCs important to safety; (6) the potential effects of seismic and non-seismic information on the postulated design bases and how they relate to a surge and seiche in the vicinity of the site and the site region; (7) any additional information requirements prescribed in the "Contents of Application" sections of the applicable subparts of 10 CFR Part 52.

2.4S.5.2 Summary of Application

In Section 2.4S.5, the applicant addresses the information related to probable maximum surge and seiche flooding in terms of impacts on structures and water supply. In addition, in this section, the applicant provides supplemental information to address COL License Information Items 2.14 and 3.5 identified in DCD Tier 2, Revision 4, Section 2.3.

The applicant addressed these issues as follows:

COL License Information Items

- COL License Information Item 2.14 Floods

COL License Information Item 2.14 requires COL applicants to provide site-specific information related to historical flooding and the potential for flooding at the plant site, including flood history, and flood design considerations.

- COL License Information Item 3.5 Flood Elevation

COL License Information Item 3.5 requires COL applicants to ensure that the design-basis flood elevation for the ABWR standard plant structures will be 30.5 cm (12 in.) below grade. This information is provided below.

2.4S.5.3 Regulatory Basis

The relevant requirements of the Commission regulations for consideration of the effects of probable maximum surge and seiche, and the associated acceptance criteria, are in Section 2.4.5 of NUREG–0800.

The applicable regulatory requirements for identifying surge and seiche hazards are as follows:

- 10 CFR Part 100, as it relates to identifying and evaluating hydrological features of the site. The requirement to consider physical site characteristics in site evaluations is specified in 10 CFR 100.20(c).

- 10 CFR 100.23(d)(3), as it sets forth the criteria to determine the siting factors for plant design bases with respect to water levels and wave action at the site

- 10 CFR 52.79(a)(1)(iii), as it relates to identifying hydrologic site characteristics with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

In addition, the staff used the regulatory positions of the following regulatory guides for the identified acceptance criteria:

- RG 1.27, "Ultimate Heat Sink for Nuclear Power Plants"

- RG 1.59, "Design Basis Floods for Nuclear Power Plants," as supplemented by best current practices

- RG 1.102, "Flood Protection for Nuclear Power Plants"

2.4S.5.4 Technical Evaluation

The staff reviewed the information in Section 2.4S5 of the STP Units 3 and 4 COL FSAR. The staff's review confirmed that the information in the application addresses the probable maximum surge and seiche flooding. The staff's technical review of this section includes an independent review of the applicant's information in the FSAR and in the responses to the RAIs. The staff supplemented this information with other publicly available sources of data.

This section describes the staff's evaluation of the technical information presented in FSAR Section 2.4S.5.

<u>COL License Information Items</u>

COL License Information Item 2.14	Floods
COL License Information Item 3.5	Flood Elevation

The staff reviewed the applicant's information in FSAR Section 2.4S.5. The staff found the methods and tools used in conjunction with or developed using this information to be reasonable. This section considers the following:

- inundation of the STP Units 3 and 4 site from a probable maximum storm surge (PMSS)
- effects of PMSS inundation on the main cooling reservoir embankment

2.4S.5.4.1 Probable Maximum Winds and Associated Meteorological Parameters

Information Submitted by Applicant

The applicant establishes the probable maximum meteorological winds (PMMWs) using guidance found in NOAA NWS Report 23 (NOAA, 1979). A summary of the applicant's PMMW parameters is provided in Table 2.4S.5-1 below. These values are reported in FSAR Section 2.4S.5.1 and Table 2.4S.5-2 (STPNOC, 2007).

Table 2.4S.5-1. Parameters of Probable Maximum Meteorological Winds

Parameter (units)	Symbol	Range of Values
Peripheral pressure (cm/in. of Hg)	P_w	76.50 / 30.12
Central pressure (cm/in. of Hg)	P_o	66.52 / 26.19
Pressure differential (cm/in. of Hg)	$P = P_w - P_o$	9.98 / 3.93
Radius of maximum winds (nautical miles)	R	5 to 21
Forward speed (knots)	T	6 to 20
Hg = mercury; in. of Hg = one-thirtieth of atmospheric pressure (e.g., 0.49 psia).		

Using the above characterization of the PMH, and following the guidance of NWS Report 23 (NOAA 1979), the applicant estimates that the PMMW speed range for a stationary hurricane is 68.0 to 71.5 m/s (152 to 160 mph).

NRC Staff's Technical Evaluation

The staff used NWS 23 (NOAA, 1979) to independently estimate the PMMW for the STP site. The staff's estimates of the PMH parameters using NWS guidance (Jelesnianski et al., 1992) are given in Table 2.4S.5-2 below.

Based on these values, the staff estimated that the maximum wind speed for a moving and a stationary hurricane at the STP site would be approximately 70.5 and 66.9 m/s (157.6 and 149.7 mph), respectively. The staff-estimated stationary hurricane wind speed of 149.7 mph is consistent but slightly lower than the applicant's estimated range of 68.0 to 71.5 m/s (152 to 160 mph) reported in FSAR Table 2.4S.5-3 (STPNOC, 2007).

The applicant initially used the SURGE and the NOAA Sea, Lake, and Overland Surges from Hurricanes (SLOSH) models to analyze storm surges. The staff issued RAI 02.04.05-1 requesting the applicant to provide the SURGE code and input and output files used to estimate the PMSS at the coast near Matagorda, Texas. The applicant responded in a letter dated June 26, 2008 (ML081970231). The staff performed an independent analysis using the applicant's implementation of the SURGE model and confirmed the applicant's analysis. Therefore RAI 02.04.05-1 is closed.

Table 2.4S.5-2. The Staff's Estimates of PMH Parameters

Parameter (units)	Value	Source in NOAA (1979)
Latitude (degrees North)	28.6	
Coriolis parameter f (1/s)	7.1×10^{-5}	
Coastal distance (km / nautical mile)	601.9 / 325	Figures 1.1 and 1.2
Central pressure P_o (cm / in. Hg)	66.52 / 26.19	$P_w - \Delta P$
ΔP (cm / in. Hg)	9.98 / 3.93	Figure 2.3
Peripheral pressure P_w (cm / in. Hg)	76.5 / 30.12	Section 2.2.2
Radius of maximum winds R (mi)	8-33.8 / 5-21	Figure 2.5
Forward speed T (m/s / knot)	3.1-10.3 / 6-20	Figure 2.7
Direction (degrees clockwise from North)	85-190	Figure 2.9
Coefficient K	79.5	Figure 2.11
Moving hurricane gradient velocity (m/s / mph)	70.5 / 157.6	Equation 2.2
Stationary hurricane gradient velocity (m/s / mph)	66.92 / 149.7	Equation 2.4

The staff issued RAI 02.04.05-4 requesting the applicant to explain (1) how NOAA's SLOSH Maximum of Maximum (MOM) water-level predictions were extrapolated to account for the PMH conditions; (2) whether the PMH used in this extrapolation was the same as the PMH used in the SURGE analysis to estimate the PMSS at the coast near Matagorda, Texas; and (3) how the applicant verified that the extrapolation is valid and conservative. The applicant responded to RAI 02.04.05-4 in a letter dated September 10, 2008 (ML082560248).

For item 1, the applicant describes how it uses the SLOSH MOM water levels to extrapolate to the PMH condition using a third-order polynomial curve fit. The applicant uses the NOAA pre-computed Categories 1 through 5 SLOSH MOM values with the corresponding pressure differentials in the curve fit. The applicant estimates the surge from the PMH using the difference between the peripheral and the central pressures as the predictor variable in the polynomial equation. The applicant's response provides the curve-fit procedure and describes its use. The staff verified the applicant's results using the PMH pressure differential.

For item 2, the applicant verifies that the conditions used for the SURGE application are consistent with those described in FSAR Subsection 2.4S.5.1. However, the applicant differentiates its application of SURGE with the use of the extrapolation based on the SLOSH MOM water levels, which differ in terms of the hurricane forward speed and the radius to the maximum winds. The applicant's assessment maintains that these differences are not important.

For item 3, the applicant's assessment of the conservatism of the SLOSH extrapolation is based on the fact that the extrapolated value is larger than a similar assessment made using the SURGE model. Also, the applicant states that NUREG–0933 refers to the SURGE as a conservative model.

The staff reviewed the applicant's response and determined that the applicant's extrapolation based on the NOAA pre-computed the SLOSH Categories 1 through 5 MOM may not yield conservative estimates of peak water levels at the site, because there is no physical basis for choosing the extrapolation equation that the applicant uses. Therefore, the staff independently estimated the PMH water surface elevations at the STP site using the SLOSH model and found that the surge level simulated by the SLOSH model is higher than the applicant's initial SURGE model estimate. Therefore, the staff issued RAIs 02.04.05-10 and 02.04.05-11. In response to RAIs 02.04.05-10 and 02.04.05-11, dated July 27, 2010 (ML102100047), the applicant performed simulations using the SLOSH and the USACE Advanced Circulation (ADCIRC) models. A summary of the applicant's analyses and the staff's subsequent review is described in Section 2.4S.5.4.2 below. Therefore RAI 02.04.05-4 is closed.

2.4S.5.4.2 Surge and Seiche Water Levels

Information Submitted by Applicant

The ABWR DCD Section 2.1 requires that the design-basis flood elevation shall be no greater than 0.3 m (1 ft) below site grade; the site grade is 10.4 m (34 ft) MSL. The applicant's estimate of the water surface elevation resulting from a PMH is 9.5 m (31.1 ft) MSL, which is lower than the design-basis flood level of 12.2 m (40 ft) MSL.

The applicant estimates the PMH using NWS Report 23 (NOAA 1979) as described in Subsection 2.4S.5.4.1 above. The applicant's procedure accounts for several factors that control the PMSS water surface elevation, but it does not include an initial sea-level rise and the astronomical tide levels associated with the PMH. The applicant added these initial sea levels separately to the estimated water levels. The applicant uses an initial sea-level rise of 0.73 m (2.4 ft) and a 10 percent exceedance astronomical high tide of 0.67 m (2.2 ft).

The applicant describes historical hurricane surge elevations along the Texas coastline. The applicant states that the peak storm surge elevation for a site close to STP Units 3 and 4 is approximately 4.9 m (16 ft) MSL.

The applicant initially uses two approaches to estimate the storm surge flooding elevations near the STP site. The first approach uses the SURGE model (Bodine, 1971) to estimate the storm surge at the Gulf coast near the STP site. The applicant's analysis examines a range of values for wind and bottom frictions, PMH geometries, and track speeds. The applicant increases the maximum SURGE estimates at the coast to 6.1 m (20.04 ft) MSL to account for the sea-level rise of 0.59 m (1.93 ft) due to global climate change. To estimate the storm surge level near the STP site, the applicant uses both the HEC-RAS model and the SURGE result to specify boundary conditions for a Colorado River backwater calculation. The HEC-RAS model simulates the combined effect of a 100-year river flood event combined with the SURGE results.

In the second approach, the applicant extrapolates archived results from NOAA's SLOSH model (Jelesnianski et al., 1992) runs that use several hurricane scenarios involving Category 1 through Category 5 hurricanes to account for PMH conditions near the STP site. NOAA reported the maximum water surface elevations from the suite of SLOSH runs in this archive. The archived SLOSH results included a 0.6-m (2.0-ft) rise in the simulations. Although the archived SLOSH results cover a range of hurricanes, the most extreme of these is weaker than the PMH. None of the archived SLOSH results indicates the inundation of the STP site. Therefore, the applicant extrapolates these SLOSH results to estimate the PMH water surface elevations, which includes the aforementioned 0.6-m (2.0-ft) offset. The applicant makes adjustments to this surge elevation to account for a long-term sea-level rise, an initial sea-level rise, and astronomical tides.

The resulting water surface elevations, the site grade elevation, and the ABWR DCD site parameter are as follows:

- HEC-RAS backwater using the SURGE water level: 7.4 m (24.29 ft) MSL.
- SLOSH extrapolation using Categories 1 through 5 estimates yields a surge water level of 8.3 m (27.2 ft) MSL.
- The consideration of a large value (10 percent exceedance astronomical tide), sea-level rise, and atmospheric pressure correction adds 1.4 m (4.53 ft) and yields a peak surge estimate of 9.7 m (31.7ft) MSL.
- The site grade elevation is 10.4 m (34.0 ft) MSL.
- The ABWR DCD compliance elevation is 10.1 m (33.0 ft) MSL.

During the site audit conducted on August 31 and September 1, 2010, the applicant presented a summary of the analyses that used the NOAA NWS SLOSH model and the USACE ADCIRC model to estimate the PMSS at the STP site. On the basis of the applicant's presentation at the site audit, the staff determined that the applicant has not shown that the ADCIRC model results account for the most conservative and plausible PMH scenario because, at that time, the applicant had only simulated one PMH scenario using the ADCIRC model. Furthermore, the descriptions and results of these model applications were not in the FSAR updates.

After the site audit, the staff issued Supplemental RAI 02.04.05-11 requesting the applicant to provide additional information regarding (1) a detailed description of the ADCIRC model, including the wind-wave submodel; (2) a detailed description of supporting data sets, including the topographic and bathymetric grids; (3) a list of conservatively selected plausible PMH scenarios consistent with the NWS 23 ranges of the PMH parameters used as inputs to the ADCIRC; (4) a description and justification of why other plausible PMH scenarios were not selected as conservative; (5) a description of the sensitivity of the ADCIRC-simulated PMSS to the PMH parameters including the radius to maximum winds, forward speed, track direction, and the landfall location; (6) a description of nonlinearity in the estimated PMSS corresponding to various combinations of PMH parameters; (7) the selected PMSS near the STP site, including the wind-wave runup; (8) a detailed description of various methods used to estimate current velocities during a PMSS event; (9) a detailed description and justification of the simplifying assumptions; (10) conservatively selected current velocities and the durations that these currents will affect the main cooling reservoir embankment; and (11) relevant citations to support a justification for the ability of the grass-lined outer face of the northern main cooling reservoir embankment to withstand the current velocities without erosion severe enough to cause an embankment breach. The RAI response dated November 22, 2010 (ML103330369), states that the applicant had performed ADCIRC simulations in addition to the scenario presented to the staff during the site audit on August 31 and September 1, 2010.

The applicant's response to RAI 02.04.05-11 part (1) describes the ADCIRC model. The applicant states that the ADCIRC is a hydrodynamic circulation model that simulates water levels and current over an unstructured domain. The model is capable of a two- or three-dimensional representation of hydrodynamics using equations of motion for a moving fluid over the surface of the rotating earth. The model uses finite element and finite difference formulations for discretizations in space and time, respectively. The applicant states that the ADCIRC can handle a variety of boundary conditions, including external and internal barrier overflow and the outward radiation of waves. The unstructured computational grid allows for smaller grid elements in areas where greater spatial resolution is necessary to capture topographical variations or to accurately capture rapid changes in hydrodynamics. The model also allows for a variation in friction with the depth of flow. The applicant states that the spatially varying friction was used for low-velocity deeper offshore waters, shallow near-shore waters, rivers and inlets where velocities are expected to be higher, and in the remaining areas of the domain. The model also represents the wetting and drying of the grid elements based on computed depths at all nodes of a grid element. The model includes only wet elements, with all nodes simulated to have a positive water depth in the solution. The model uses a minimum water velocity as a criterion for determining whether water can flow from an adjacent wet element to a dry one.

The applicant states that the ADCIRC model uses the asymmetric Holland wind model (Holland 1980). The applicant uses the USGS National Land Cover Data Classification map and land roughness lengths derived from the Federal Emergency Management Agency (FEMA) HAZUS software program, which is used to assess hazard losses—including those from hurricanes. The roughness of an inland grid element changes as the element becomes inundated during the hurricane event. The applicant carried out an extensive validation of the ADCIRC predictions on the Texas coastline for historical hurricanes. The applicant notes that these validation studies included Hurricanes Rita and Ike, which produced large storm surges; accurate measurements of hurricane properties and surge levels for these storms are available.

The applicant states that the ADCIRC uses the computer program SWAN to estimate the wind setup. The Delft University of Technology developed the SWAN program, which computes random, short-crested, wind-generated waves in near-shore and inland waters. The SWAN model accounts for wave propagation, shoaling, reflection, refraction, frequency shifting, wave interactions, white capping and breaking, and dissipation. The applicant states that water levels and currents are computed by the ADCIRC with input into SWAN, which recalculates the water depth to account for the wave processes. The ADCIR model further uses the modified hydraulic properties computed by SWAN.

The applicant also states that along the coastal areas of the United States, FEMA has certified the ADCIRC for use in the development of Flood Insurance Rate Maps that need to account for flooding from storm surges. The applicant also notes that the ADCIRC is the standard coastal model used by the USACE.

In the response to RAI 02.04.05-11 part (2), the applicant provides a detailed description of data sets used with the ADCIRC. The applicant states that topographic data used in the ADCIRC are the most accurate and current. The applicant also states that the most accurate topographic data are derived from the Light Detection and Ranging (LIDAR) data sets from the Texas Water Development Board (TWDB) (Harris County Flood Control District, FEMA, Louisiana State University) and the Louisiana Oil Spill Contingency Office Atlas. The applicant states that the LIDAR data were initially available at a 10-m resolution and later at a 1-m resolution; the data include small-scale features such as levees, riverbanks, and roads. The ADCIRC computational grid was initially built using the 10-m LIDAR data and was later refined using the 1-m LIDAR data to include hydraulically relevant features. The alignment of major topographic features including roads, shorelines, and rivers was checked against aerial photographs and satellite images.

The applicant states that Texas topographic grid Version 13 (or the TX2008 model) incorporates the western North Atlantic Ocean, the Gulf of Mexico, the Caribbean Sea, and the Texas coastal floodplains to allow full dynamic coupling between oceans, continental shelves, and coastal floodplains. The applicant states that the TX2008 model domain's eastern boundary is the open ocean that lies along the 60°W meridian. The open ocean boundary (1) is located in deep ocean, (2) lies outside of any resonant basins, (3) is geometrically simple, (4) has limited nonlinear energy because of the depth, and (5) its tidal response is mainly determined by astronomical variations. The applicant states that the specification of a boundary condition along this open ocean boundary is simple because the hurricane storm surge response along it is mainly an inverted barometric pressure effect directly correlated to the hurricane pressure field.

The applicant also states that the TX2008 model domain is bounded by the land boundary of the eastern coastlines of North, Central, and South America. The highly detailed region represented in the TX2008 model extends from Brownsville to Port Arthur, Texas; the TX2008 model extends inland and runs along the 9- to 23-m (30- to 75-ft) elevation contour. The model incorporates the Brazos, Nueces, and Rio Grande rivers and major dredged navigation canals such as the Gulf Intracoastal Waterway; all significant levee systems, elevated roads, and railroads are barrier boundaries. The applicant notes that the grid resolution in the TX2008 model varies from 19 to 24 km (12 to 15 mi) in deep ocean and about 30 m (100 ft) in near-shore areas of Texas.

The applicant also states that the bathymetric data for the western North Atlantic, Gulf of Mexico, and Caribbean Sea were derived from the raw bathymetric sounding database from the NOAA National Ocean Service Digital Nautical Charts database and NOAA ETOPO5 data. The bathymetry for inland waterways in coastal regions of Texas was derived from regional bathymetric and dredging surveys from the USACE, NOAA, TWDB, or nautical charts. The geometry, bathymetry, and topography in the TX2008 model represent post-Hurricane Ike conditions.

The applicant states that the ADCIRC computational grid should account for pronounced vertical features that are small in the horizontal scale compared to the grid spacing. Some of these features can be a significant obstruction to the flow. Therefore, features higher than 3 m (10 ft) from the surrounding area were carefully incorporated into the model as subgrid scale weirs or lines of nodes with crown elevations.

The response to RAI 02.04.05-11 part (3) states that the applicant used combinations of three landfall points and NWS 23 PMH parameters—radius to maximum winds, approach direction, and forward speed—to specify 81 PMH scenarios that may occur at the STP site. The applicant states that NWS 23 ranges of PMH parameters near the STP site include a radius to maximum winds of 9.7 to 33.5 km (6.0 to 20.8 mi), an approach direction of 97.5 to 190 degrees clockwise from the north, and a forward speed of 11.1 to 35.1 km/hr (6.9 to 21.8 mph). The applicant notes that storm surge simulations using SLOSH indicate that the maximum water surface elevation near STP Units 3 and 4 would be produced by a PMH scenario with a large radius to maximum winds, fast forward speed, and prevailing winds blowing from the east toward the site. The applicant concludes that the PMH would result from a storm with a radius to maximum winds of 33.5 km (20.8 mi), an approach angle of 143.8 degrees clockwise from the north, and a forward speed of 35.1 km/hr (21.8 mph, 18.9 kt).

The applicant postulated a series of hurricane scenarios using the ADCIRC to determine the maximum water surface elevation at the STP Units 3 and 4 site. The applicant used a radius to maximum winds of 38.6 km (24 mi, 21 nm); an approach direction of 135 degrees clockwise from the north; a forward speed of 37 km/hr (23 mph, 20 kt); a central pressure of 887 mb (26.19 in. Hg); and a peripheral pressure of 1,020 mb (30.12 in. Hg). The only variables were the distance of the storm track from the site and the track's direction. The applicant used seven ADCIRC scenarios (summarized in Table 2.4S.5-3 below). In the response to RAI 02.04.05-10 (ML102100047), the applicant states that the initial conditions for the ADCIRC runs consisted of a water surface elevation that accounted for a 10 percent exceedance high tide, initial rise, and long-term sea-level rise estimated by NOAA.

Table 2.4S.5-3. The Applicant's PMH Scenarios for ADCIRC Simulations

Scenario	Distance from Site	Track Direction	Maximum PMSS Water Surface Elevation
1	19.3 km (12 mi, 10.4 nm)	NW	8.1 m (26.5 ft) MSL
2	38.6 km (24 mi, 20.9 nm)	NW	8.9 m (29.3 ft) MSL
3	57.9 km (36 mi, 31.3 nm)	NW	8.7 m (28.5 ft) MSL
4	38.6 km (24 mi, 20.9 nm)	N	7.6 m (25 ft) MSL
5	38.6 km (24 mi, 20.9 nm)	N-NW	8.8 m (29 ft) MSL
6	38.6 km (24 mi, 20.9 nm)	W-NW	7.9 m (26 ft) MSL
7	38.6 km (24 mi, 20.9 nm)	W	6.1 m (20 ft) MSL

The applicant also states that the ADCIRC simulations use the same wind profile that the SLOSH uses because the SLOSH wind profile results in greater wind speeds than in the Holland profile for the same gradient wind speed and distance from the storm's center.

The response to RAI 02.04.05-11 part (4) states that the applicant selected PMH scenarios that represent the most conservative combination of storm scenarios, because the selected storm scenarios use the greatest ΔP that results in a stronger storm, the greatest radius to maximum winds (Scenario 2) that results in a larger storm, the greatest forward speed that increases surge heights, maximum sustained wind speed that remains constant until landfall, tracks that are least resistant to wave build-up, and a conservative wind profile.

In the response to RAI 02.04.05-11 part (5), the applicant reports the maximum surge heights predicted by the ADCIRC for the seven PMH scenarios. These maximum surge heights are listed above in Table 2.4S.5-3. The applicant notes that the ADCIRC did not successfully simulate scenario 7. The applicant estimated the surge water surface elevation at the site for scenario 7 based on a completed ADCIRC simulation that used a lower wind speed and an estimate of the incremental surge expected for the difference in wind speed. Based on the ADCIRC-simulated maximum water surface elevation at the site, the applicant concludes that the greatest storm surge occurs when the storm passes the site at a distance equal to the radius of maximum winds and the storm track direction is generally to the northwest. In a comparison of topographical data used in the SLOSH and ADCIRC, the applicant notes that the TX2008 model accounts for pronounced vertical features with a small horizontal extent like the levee surrounding the City of Matagorda and the Gulf Intracoastal Waterway.

In the response to RAI 02.04.05-11 part (6), the applicant states that to a limited degree, surge elevations do not vary linearly with track direction or distance from the site. The applicant also states that it was difficult to describe the nature of the nonlinearity, although the outcomes were consistent with the behavior of hurricane storm surges in the western Gulf of Mexico.

The applicant's response to RAI 02.04.05-11 part (7) states that based on ADCIRC simulations using the SLOSH wind profile, the estimated PMSS at the STP Units 3 and 4 site is 8.9 m (29.3 ft) MSL. The applicant also states that this PMSS would occur as a result of a hurricane traveling in a northwestern direction and passing within 38.6 km (24 mi) of the site. Until landfall, the hurricane would have a constant speed of 37 km/hr (23 mph), a central pressure of 887 mb, and a maximum sustained wind speed of 296 km/hr (184 mph, 160 kt). The hurricane's strength would gradually decay after landfall.

The applicant's responses to RAI 02.04.05-11 parts (8) through (11) are relevant to the staff's review in Section 2.4S.10 of this SER, which is where the applicant's responses and the staff's review are summarized.

NRC Staff's Technical Evaluation

The staff issued RAI 02.04.05-1 requesting the applicant to provide the input and output files of the HEC-RAS analysis to estimate backwater effects corresponding to the PMSS estimates using the SURGE model. The applicant responded in a letter dated June 26, 2008 (ML081970231), and provided the input and output files. The staff did not use these files because the staff's independent analysis of the PMH storm surge estimate using the SLOSH model is more conservative. Therefore RAI 02.04.05-1 is closed.

The staff issued RAI 02.04.05-2 requesting the applicant to explain why a wind-stress correction factor of 1.1 was used when, as stated in FSAR Subsection 2.4S.5.2.3.1, page 2.4S.5-4, "the stresses introduced into the air by the drops can be 10-20% of the wind stress." The applicant responded in a letter dated August 27, 2008 (ML082490086), and states that the wind-stress factor is consistent with RG 1.59. The staff determined that the applicant's response is satisfactory. Therefore RAI 02.04.05-2 is closed.

The staff issued RAI 02.04.05-3 requesting the applicant to explain why the HEC-RAS backwater analysis was not carried out for the LRS through the Palacios Bay. The applicant responded in a letter dated August 27, 2008, and states that because the LRS is tidal, with no upstream inflow, and assumed to be inundated by a PMH surge, no backwater calculations were warranted. The staff agreed with the applicant's assessment. The staff's independent PMH storm surge estimate using the SLOSH model showed that the Palacios Bay would be completely inundated during the PMH event. Therefore RAI 02.04.05-3 is closed.

The staff issued RAI 02.04.05-5 requesting the applicant to explain why the PMH determined from the NOAA NWS 23 report was not used as input to run the SLOSH model to estimate water surface elevations for the PMSS. The applicant responded in a letter dated September 4, 2008 (ML082530449), and states that the SLOSH model was not available publicly or commercially to conduct an analysis specific to the PMH. The applicant's PMSS assessment is based on the NOAA precomputed SLOSH simulations for Categories 1 through 5 hurricanes. The applicant uses a third-order polynomial equation to estimate a relationship between the storm surge water surface elevation and the hurricane central pressure difference, as described above. Therefore RAI 02.04.05-5 is closed.

The staff issued RAI 02.04.05-6 requesting the applicant to indicate whether any effort was made to adjust the estimated PMH parameters in light of more recent hurricanes that have occurred since the publication of the NOAA NWS 23 report. The applicant responded in a letter dated September 4, 2008, and refers to a recent analysis by NOAA indicating that the period between 1945 and 1970 is considered a hurricane period that was as active as hurricane periods in the most recent decades. The applicant concludes that because the 1945 through 1970 period is covered by the analysis in the NWS 23 report, a PMH estimated by the NWS 23 will provide a conservative assessment and will account for any increase in hurricane strength due to future climate variability. The staff used other currently available information to assess the relative severity of the NWS 23 PMH, as described below. Therefore RAI 02.04.05-6 is closed.

The staff issued RAI 02.04.05-9 requesting the applicant to provide a physical basis to justify why the maximum of the maximum envelope of water surface elevation–ΔP relationship used by the applicant is valid, with a citation to an accepted and validated method that uses such a relationship, or provide a justification with a citation indicating why estimating parameters of a third-degree polynomial relationship from five data points would result in an accurate estimation of the model parameter values. The applicant responded in a letter dated September 16, 2009 (ML092610376). The applicant's response cited NUREG–0933, "A Prioritization of Generic Safety Issues - Item C-14: Storm Surge Model for Coastal Sites (Rev. 1)," dated 2007 and argued that a bathystrophic model, SURGE, is adequate for calculating design-basis water levels. The applicant repeated its response to RAI 02.04.05-4 regarding justification of the extrapolation equation. The staff reviewed the applicant's response and determined that the extrapolation procedure may not be conservative. Therefore, the staff performed an independent assessment using the SLOSH model to estimate the PMH inundation level at the site. The staff's approach is described below for the closure of RAI 02.04.05-9. In addition, the complex coastline, built-up areas, interacting streams, channels and canals, bathymetry, and topography near the STP site require that a more advanced method be used to accurately estimate the storm surge from severe hurricanes.

Because the staff determined that the applicant's PMSS estimates using the SURGE model and the extrapolation approach may not be conservative, the staff carried out independent SLOSH simulations using a range of input values that represent the variability of PMH conditions at the STP site. The NWS 23 specifies ranges of PMH parameters, radius to maximum winds (9.7 to 33.5 km [6 to 20.8 mi]), the approach direction (97.5 to 190 degrees clockwise from the north), and the forward speed of the storm (3.1 to 9.8 m/s [6 to 19 knots]). The staff used combinations of these three parameters in addition to three different landfall points to specify several PMH scenarios that may occur at the STP site. Three individual values were selected for each of these scenarios and, therefore, the staff's analysis resulted in the SLOSH simulation of a total of 81 PMH storm tracks.

The staff set the radius to maximum winds to 9.7, 20.8, and 33.5 km (6, 12.9, and 20.8 mi); the approach angle to 97.5, 143.8, and 190 degrees clockwise from the north; and the forward speeds to 3.1, 6.4, and 9.8 m/s (6, 12.5, and 19 knots) for each run. The three landfall points were selected so that the first landfall point was located at a distance equal to the radius of the maximum winds, west of the mouth of the Colorado River Navigation Channel at the barrier islands; the second point was centered on the mouth of the Colorado River Navigation Channel at the barrier islands; and the third was located a distance equal to the radius of the maximum winds east of the mouth of the Colorado River Navigation Channel, at the barrier islands. All storm tracks are straight. There are 81 combinations of these parameters, as stated before.

The PMH storm tracks used in the staff's independent analysis are shown in SER Figures 2.4S.5-1, 2.4S.5-2, and 2.4S.5-3. These figures are differentiated by track direction. Each figure shows nine tracks (three sets of different radii to maximum winds). In these figures, the central track is represented by the solid line, and the corresponding tracks to the west and the east are represented by matching broken lines.

A vertical datum offset is assigned in the SLOSH model to account for tides or other factors that cause the baseline sea level to be other than 0 ft NGVD29. The staff cites this parameter to include the following:

- a 10 percent exceedance for astronomical tides (0.67 m [2.2 ft]) above the mean low water [MLW]) taken from RG 1.59 for Freeport, Texas (Table C.1)
- an initial rise of 0.73 m (2.4 ft) taken from RG 1.59 for Freeport, Texas (Table C.1)
- a 100-year sea-level rise of 0.44 m (1.43 ft) taken from the NOAA tide gauge at Freeport, Texas (NOAA 2009)

Therefore, the staff used a datum offset of 1.84 m (6.03 ft) above the MLW when accounting for 100 years of sea-level rise and a datum offset of 1.4 m (4.6 ft) for present-day conditions. The MLW is 0.3 m (1.1 ft) below NGVD29 at the Freeport, Texas, tide gauge location. The vertical datum for the SLOSH is NGVD29 and therefore the staff used vertical offsets of 1.5 m (4.93 ft) (when accounting for sea-level rise) and 1.1 m (3.5 ft) for present-day conditions. The 81 PMH storm tracks were simulated for both of these cases in the staff's independent analysis.

The SLOSH simulations indicated that the maximum storm surge water surface elevation near the STP Units 3 and 4 site would be produced by a large (in terms of radius to maximum winds), fast-moving (in terms of forward speed) storm that would produce prevailing winds blowing from the east toward the STP Units 3 and 4 site. The staff prepared a map of maximum water surface elevation on the SLOSH computational grid from these 81 PMH storm-track simulations for the two sea-level rise scenarios (Figure 2.4S.5-4). As expected, the staff's analysis obtained higher water surface elevations when the long-term sea-level rise was included before initiating the SLOSH simulations..

Because hurricanes rotate counter clockwise in the northern hemisphere, the highest surges are expected on the east side of the hurricane eye due to the fastest onshore wind being toward the right of the eye. Also, topographic highs provide some protection to land areas downwind from them and conversely lead to higher surge level to land areas upwind of them. Storms with larger forward speeds generate faster responses in surge, leaving less time for the surge to dissipate over and around the surrounding terrain. Considering these factors, the site would be most vulnerable to flooding when the eye of the hurricane passes quickly to the west of the site on the leading edge of the storm. These expected trends are borne out in the SLOSH results.

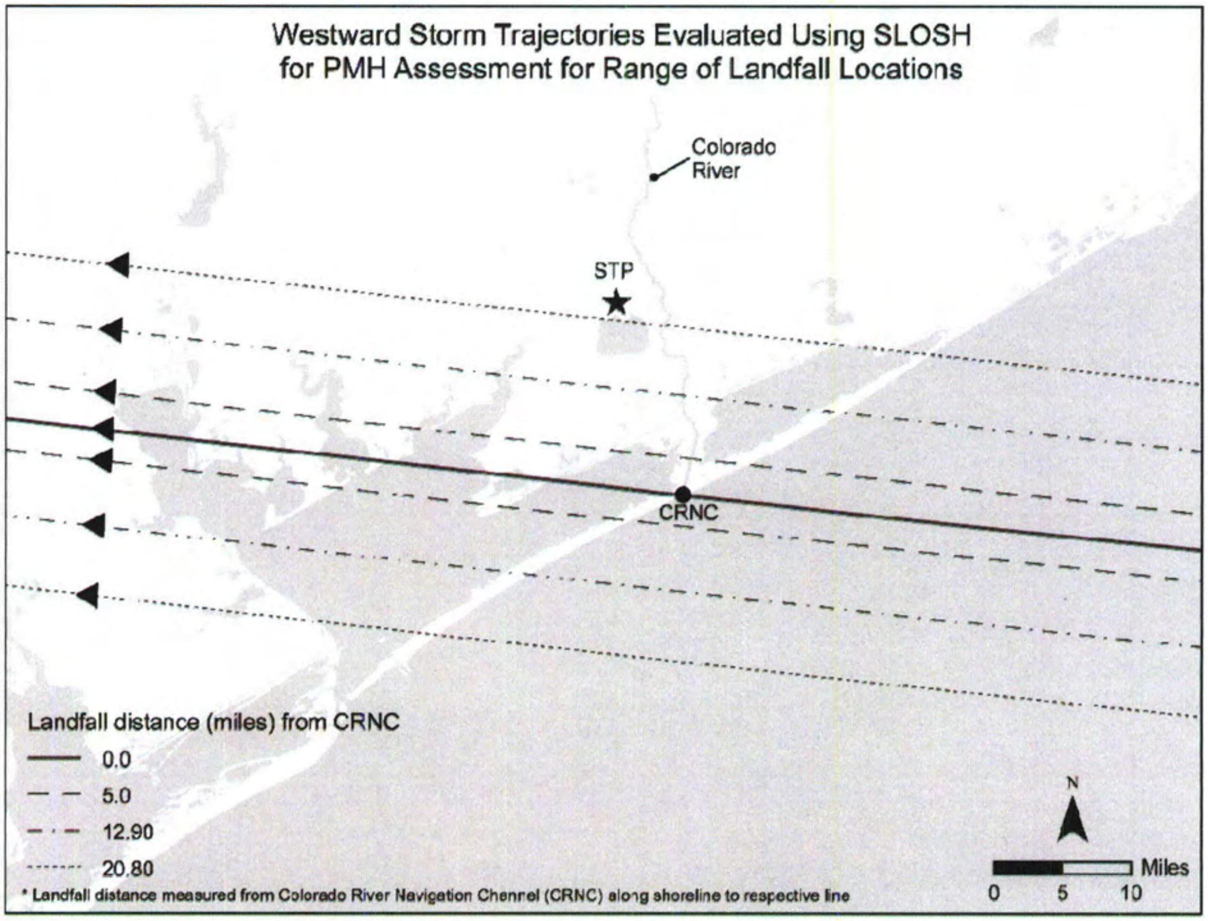

Figure 2.4S.5-1. Westward PMH Storm Tracks Used in the NRC Staff's SLOSH Simulations

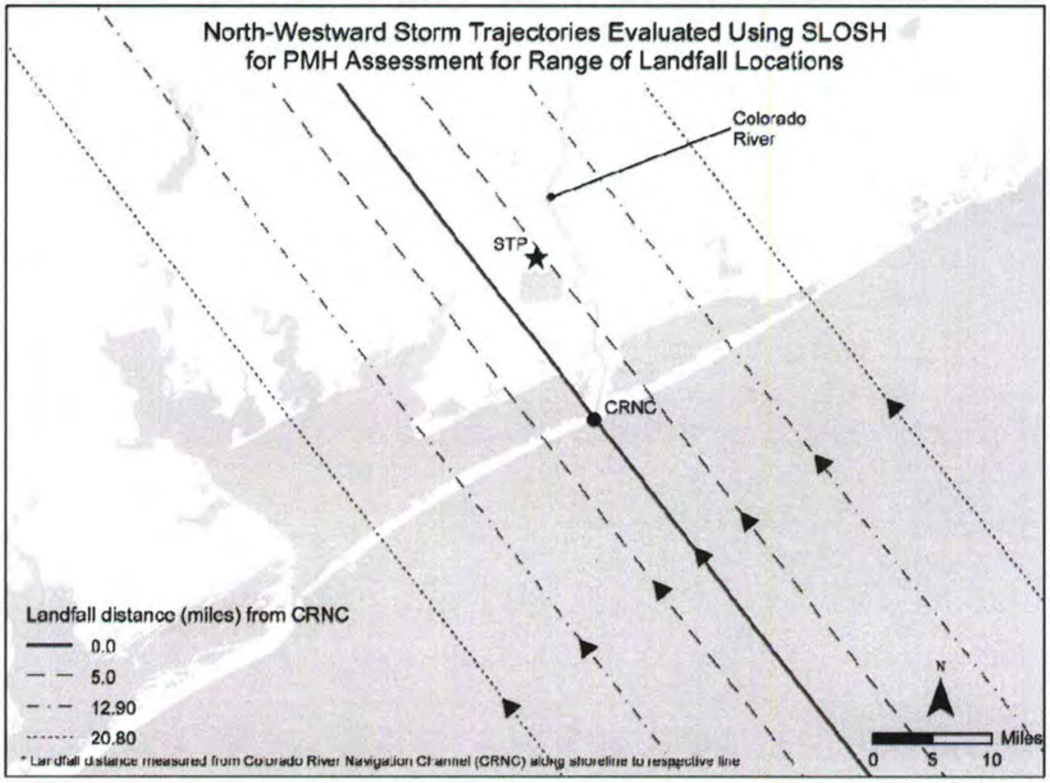

Figure 2.4S.5-2. North-Westward PMH Storm Tracks Used in the NRC Staff's SLOSH Simulations

The version of the SLOSH model used by the staff has a limitation in terms of retaining and reporting computed water surface elevations. The model truncates any water surface elevations higher than 11 m (36 ft) NGVD29 and reports the values in those grid cells as a code, which means that in any grid cell that had a value set equal to this code, the storm surge water surface elevation exceeded 11 m (36 ft) NGVD29. Because the actual values of the storm surge water surface elevation are not retained for these grid cells, the only information available at these grid cells is that the maximum water surface elevation on the grid cells exceeded 11 m (36 ft) NGVD29. The staff's simulations resulted in the STP site being inundated during the most severe of the 81 PMH scenarios simulated and the storm surge water surface elevation on the grid cell where the STP Units 3 and 4 site is located exceeded 11 m (36 ft) NGVD29. Based on values of storm surge water surface elevations at surrounding grid cells, the staff estimated that the storm surge water surface elevation at the grid cell where the STP Units 3 and 4 site is located would probably be between 11.3 and 11.6 m (37 and 38 ft) NGVD29.

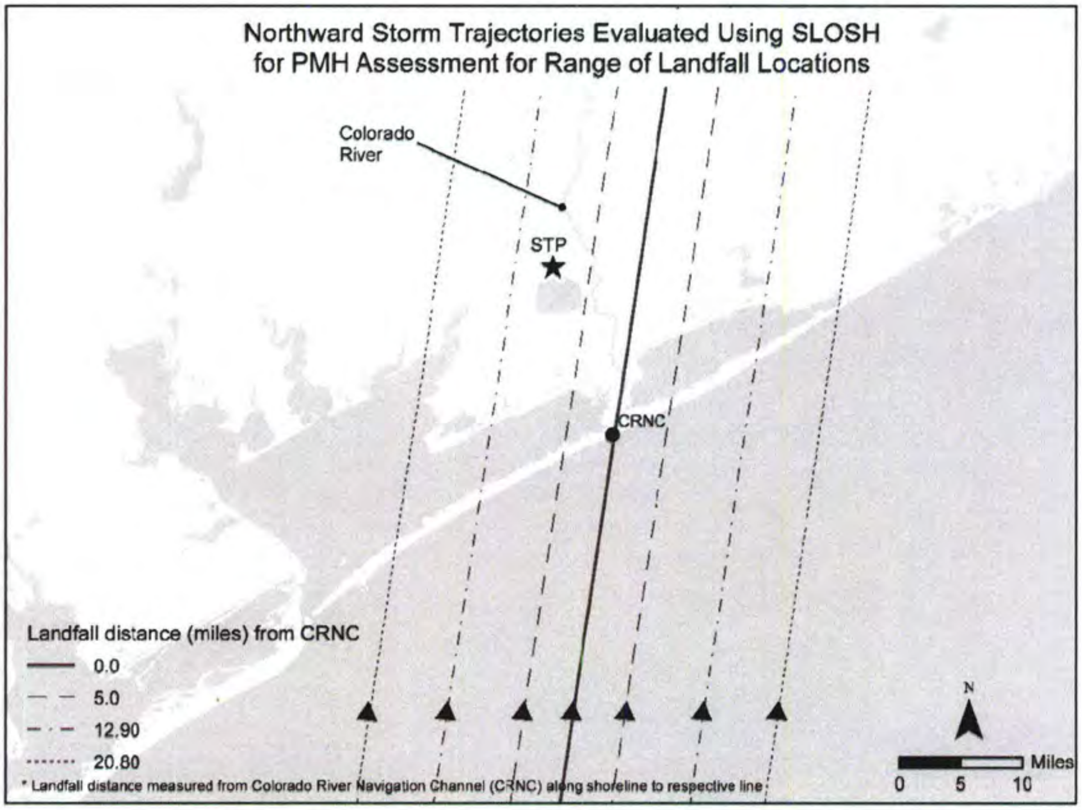

Figure 2.4S.5-3. Northward PMH Storm Tracks Used in the NRC Staff's SLOSH Simulations

The staff reviewed the applicant's responses to RAI 02.04.05-11, parts (1) through (7). The staff's independent review found that the USACE ADCIRC model has a long history of development, verification, and validation (Luettich and Westerink, 1992; Luettich et al., 1992; Westerink et al., 1992; Blain et al., 1994; Grenier et al., 1994; Westerink et al., 1994; Luettich et al., 1998; Gica et al., 2001; Dietsche et al., 2007; Demirbilek et al., 2008; Westerink et al. 2008; Funakoshi et al.., 2009; Bunya et al., 2010). The staff therefore determined that ADCIRC is an appropriate model for simulating storm surges from hurricane events. FEMA (2010) is currently using the ADCIRC model for flood insurance studies in coastal areas of the Atlantic Ocean and Gulf of Mexico.

The staff also reviewed the characteristics of bathymetry and near-shore topographic data used to represent the computational domain in the ADCIRC. The staff determined that the ADCIRC bathymetric and topographic data used by the applicant and contained in the TX2008 model are significantly more detailed than those used in the SLOSH computational basins. The more detailed ADCIRC data resolve surface features with greater detail and accuracy. Another advantage of the ADCIRC model and computational grid is the ability to include topographic features at scales smaller than the grid resolution. These features allow the hydrodynamics to be simulated with much greater fidelity in the near-shore areas, because the hurricane storm surge interacts in complex ways with coastal features such as bays, estuaries, and rivers; and with buildings, roads, and levees. Two of the features that the ADCIRC computational grid resolves with greater vertical accuracy than in the SLOSH computational basin for the

Matagorda Bay area are the City of Matagorda levee and the dredge piles along the lower Colorado River. In particular, the City of Matagorda levee lies directly in the path of a hurricane storm surge as it advances from the open waters of the Gulf of Mexico toward the STP site. The staff concluded that these features of the ADCIRC bathymetric and near-shore topographic data provide more detailed site-specific information for storm surge simulation at the STP site compared to the SLOSH model.

The staff also reviewed the applicant's statement provided in RAI response 02.04.05-11 that NWS 23 ranges of PMH parameters near the STP site include a radius to maximum winds of 9.7 to 33.5 km (6.0 to 20.8 mi), an approach direction of 97.5 to 190 degrees clockwise from the north, and a forward speed of 11.1 to 35.1 km/hr (6.9 to 21.8 mph). As described above, the staff independently obtained NWS 23 PMH parameter ranges including a radius to maximum winds of 9.7 to 33.5 km (6 to 20.8 mi), an approach direction of 97.5 to 190 degrees clockwise from the north, and a forward speed of the storm of 3.1 to 9.8 m/s or 11.1 to 35.2 km/hr (6 to 19 knots). The staff therefore concluded that the applicant has appropriately selected the PMH parameters from NWS 23.

The staff also reviewed the applicant's PMH scenarios provided in RAI response 02.04.05-11. The staff determined that the applicant-identified PMH scenario that would result in the largest storm surge at the STP site is consistent with the staff's independent SLOSH simulations described above. Based on these SLOSH results, the applicant chose to simulate seven PMH scenarios in the ADCIRC. The staff determined that these seven scenarios are reasonably plausible for the STP site because they are consistent with the recommendations of NWS 23. The staff also determined that a larger, faster-moving hurricane produces a larger surge. Because the applicant uses a radius to maximum winds that is slightly more conservative than the one identified by the staff (38.6 km [24 mi] compared to staff's value of 33.5 km [21 mi]), and uses the same values for forward speed and central pressure difference, the staff concluded that the applicant has appropriately selected a conservative PMH scenario to simulate using the ADCIRC.

The above discussion provides the basis for the staff's determination that the applicant has selected conservative PMH scenarios for estimating the PMSS at the STP site. The staff also determined, as described above, that the applicant has selected an appropriate model supported by site-specific information. Therefore, the staff concluded that the applicant's ADCIRC simulations for determining the PMSS at the STP site are adequate and RAI 02.04.05-11 is considered closed..

2.4S.5.4.3 Wave Action

Information Submitted by Applicant

The applicant determines that wave action coupled with the probable maximum surge is not the controlling wave scenario. The applicant assesses wave action coupled with flooding described in FSAR Section 2.4S.4 to be more conservative.

The applicant uses the USACE ADCIRC model to perform PMSS estimation. The ADCIRC model is tightly coupled with the SWAN model that computes wind waves within the ADCIRC-SWAN runs. The applicant states that the maximum PMSS water surface elevation of 8.9 m (29.3 ft) MSL includes wind-wave effects.

NRC Staff's Technical Evaluation

The staff conservatively estimated the maximum PMH storm surge water surface elevation to be between 11.3 and 11.6 m (37 and 38 ft) NGVD29 near the STP Units 3 and 4 site. The water depth near the site would be approximately 0.9 to 1.2 m (3 to 4 ft) at this location. For this shallow water depth, the PMH wind speeds, and unlimited fetch, the staff estimated the wind-wave amplitude to be 0.27 to 0.36 m (0.9 to 1.2 ft) following the methods in the Coastal Engineering Manual (USACE 2008). The wave runup is a function of the depth of the water and the ground slope over which the wave passes. The ground slope is not precisely known, so a range of reasonable values was used. As the ground steepens, wave runup becomes higher. Based on the conservative assumption of an armored shore, the staff used a steepest slope of 10 percent. The staff determined the corresponding conservative wave runup to be approximately 0.20 m (0.65 ft). Therefore, an evaluation of wave action shows that it adds 0.47 to 0.56 m (1.55 to 1.85 ft) to the peak level of inundation estimated by the SLOSH simulations.

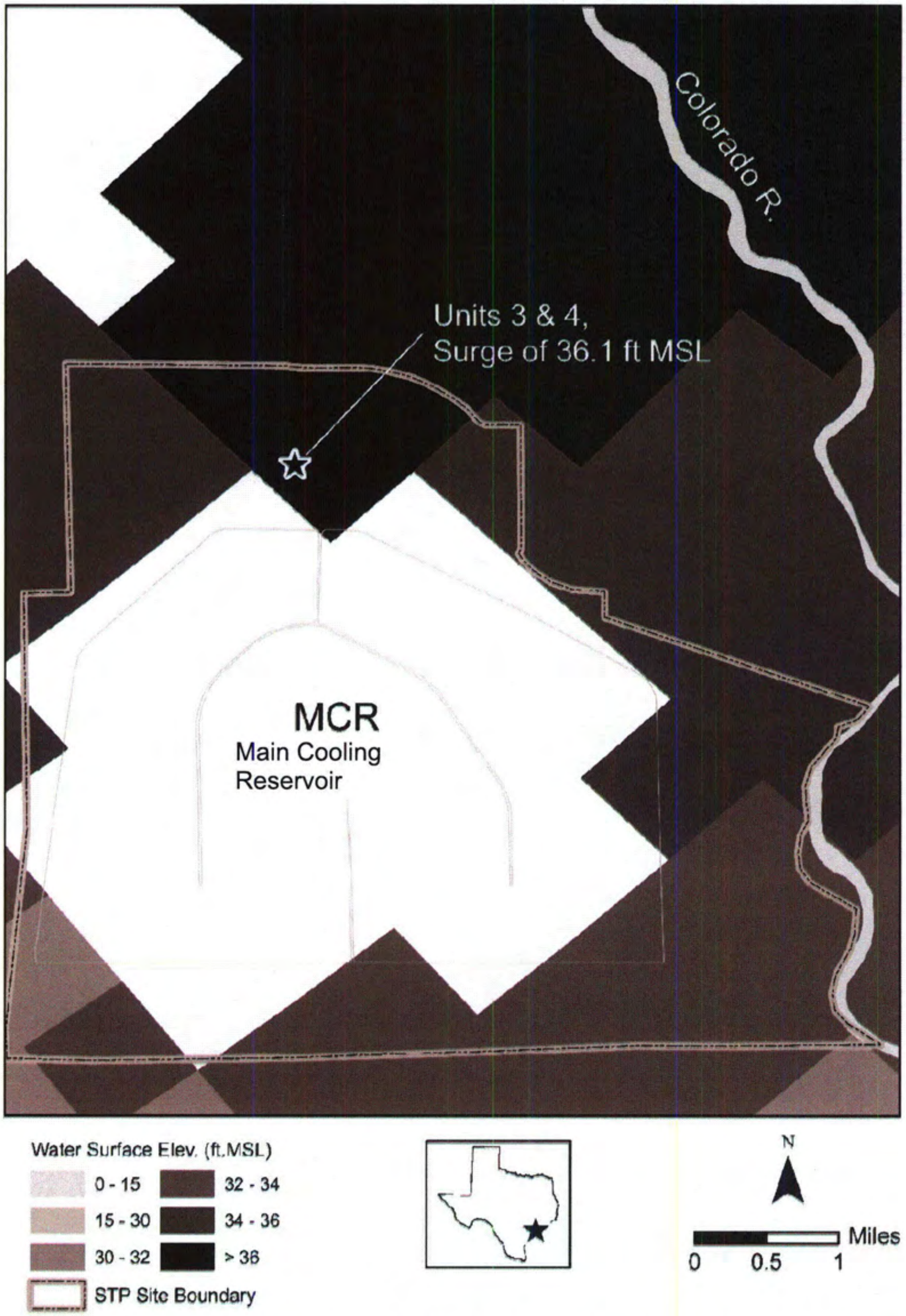

Figure 2.4S.5-4. NRC Staff-Estimated PMSS Water Surface Elevations at the STP Site

Therefore, the staff estimated the maximum PMH storm surge water surface elevation to be between approximately 11.8 to 12.2 m (38.6 to 39.6) NGVD29, including the effects of wind waves at the STP Units 3 and 4 site.

To compare the relative severity of the PMH parameters estimated from NWS 23, the staff compared them to severe storm studies that are currently being carried out (Resio 2009; Vickery 2009). The staff found that the PMH estimated by the NWS 23 method is smaller in size than those estimated near the STP site by Resio (2009), but it has greater wind speeds. On the other hand, the severe storms estimated by Vickery (2009) near the STP site are smaller in size than the PMH, but they have slightly greater wind speeds. The storm surges estimated by Resio (2009) inundate the STP site. However, the maximum stillwater surface elevations are less than those estimated by the staff's independent analysis described above. The Vickery (2009) simulations of storm surge at the STP site were also carried out using the SLOSH model, but they did not result in the inundation of the STP site.

Based on the above information, the staff concluded that the PMH estimated from the NWS 23 method provides a conservative estimate of the maximum storm surge water surface elevation at the STP site.

The staff determined that the applicant's site-specific PMSS maximum water surface elevation of 8.9 m (29.3 ft) MSL is reasonable and conservative. Although the applicant does not provide an estimate of the wind-wave runup (the wind-wave setup is accounted for in the ADCIRC simulations), the staff determined that the applicant's independent estimate of 0.20 m (0.65 ft) could be conservatively added to the applicant's PMSS stillwater and wind setup estimate, because the staff's estimate is derived from a more conservative PMSS scenario. Therefore, the staff concluded that the maximum PMSS water surface elevation at the STP Units 3 and 4 site accounting for the wind setup and runup would not exceed 9.1 m (30 ft) MSL and would be 0.6 to 1.2 m (2 to 4 ft) below the STP Units 3 and 4 site grade of 10.4 to 11 m (34 to 36 ft) MSL. Because the PMSS maximum water surface elevation accounting for wind-wave effects is below the site grade and is exceeded by the maximum water surface elevation expected during the postulated main cooling reservoir embankment breach event, the staff concluded that further investigation of the PMSS at the STP site is not warranted.

2.4S.5.4.4 Resonance

Information Submitted by Applicant

The applicant identifies no scenario that will produce resonance effects.

NRC Staff's Technical Evaluation

The applicant states that there is no scenario that would produce resonance effects. FSAR Section 2.4S.8 analyzes PMH winds as a potential mechanism for the generation of resonant seiches in the main cooling reservoir (STPNOC, 2007). Consideration of the geometry and water depths of the main cooling reservoir allows for estimates of the necessary wind-wave frequency that could lead to a seiche; the differences between the PMH wind wave and the natural resonant frequency leads to the conclusion that there is no possibility of this postulated coupling.

The staff issued RAI 02.04.05-7 requesting the applicant to provide an assessment of seismically induced seiches in the main cooling reservoir. In a letter dated August 12, 2008 (ML082270381), the applicant's response states that there was no consideration of seiche effects in the main cooling reservoir from seismic forcings. The applicant considers the main cooling reservoir embankment failure as the bounding case for site flooding and a design-basis flood for the STP Units 3 and 4 site. The staff performed an independent assessment of seismic seiche in the main cooling reservoir. Section 2.4S.8 of this SER discusses the staff's independent assessment, including resonance in the main cooling reservoir.

2.4S.5.4.5 Protective Structures

Information Submitted by Applicant

The applicant assesses the flood-level estimate from the postulated main cooling reservoir embankment breach to be the controlling event related to safety-related facilities. This analysis is discussed in FSAR 2.4S.4.

NRC Staff's Technical Evaluation

The applicant considers the flood generated by a postulated failure of the main cooling reservoir embankment to be the controlling flood at the STP Units 3 and 4 site and therefore, the design-basis flood for protecting all safety-related SSCs. The staff's independent assessment in this section indicated that the PMH storm surge would flood the STP Units 3 and 4 site. The staff also determined that the PMH storm surge would result in floodwaters surrounding the main cooling reservoir embankment. The outside face of the main cooling reservoir embankment is not protected from the effects of a flood other than a grass lining. The staff postulated an induced failure of the main cooling reservoir embankment because of the sloshing and erosive action of floodwaters surrounding the main cooling reservoir during a PMH storm surge event, as described below.

The staff reviewed the applicant's responses regarding flood-protection requirements. The staff concluded that there is one combined event scenario that the applicant did not address in the FSAR. The staff estimated the storm surge resulting from a PMH above. The staff concludes that although the PMH storm surge water surface elevation at the STP Units 3 and 4 site will not exceed the floodwater surface elevation resulting from the postulated breach of the main cooling reservoir north embankment, it may provide a trigger for the failure of the main cooling reservoir embankment. The outside surface of the main cooling reservoir embankment is lined with grass and is not protected by any riprap or armoring, which makes the embankment vulnerable to storm surge currents and erosion. The main cooling reservoir embankment is equipped with a seepage-control system consisting of a sand drain blanket and relief wells around the reservoir to protect the toe of the embankment by lowering the seepage level. However, the seepage-control system is lower than the system at the PMH surge level and will not be functioning during the surge inundation, which could trigger a main cooling reservoir embankment breach. Therefore, the staff postulated that it is possible for the main cooling reservoir embankment to fail while being under inundation during the PMH storm surge. As an example, the 2005 New Orleans flooding from Hurricane Katrina was caused by a combination of storm surge and levee failure, where the failure was caused by both seepage and overtopping (U.S. General Accounting Office [GAO] 2006). The staff therefore determined that for the STP site, a combination of the PMH storm surge and main cooling reservoir embankment failure needs to be investigated to estimate the maximum floodwater surface elevation at the STP Units 3 and 4 site.

Based on the applicant's FSAR the staff determined that the applicant has not shown that the ADCIRC model results account for the most conservative plausible PMH scenario. Furthermore, the description and result of these model applications are not included in the FSAR. Therefore the staff issued RAI 02.04.05-11, which was tracked as **Open Item 2.4.5-1**.

The applicant responded to RAI 02.04.05-11 in a letter dated November 22, 2010 (ML103330369). The staff determined that the applicant's response adequately demonstrates that the applicant has appropriately and conservatively selected PMH scenarios, simulation models, and site-specific data for estimating the PMSS at the STP site. The staff concluded that the PMSS maximum water surface elevation accounting for wind-wave effects is below the site grade and is exceeded by the maximum water surface elevation expected during the postulated main cooling reservoir embankment breach event. The staff also concluded that further investigation of the PMSS at the STP site is not warranted. The applicant also provides new and updated text that will be included in a future revision of the FSAR. Therefore, **Open Item 2.4.5-1** is closed and the proposed FSAR text changes will be tracked as Confirmatory Item 02.04.05-1.

2.4S.5.5 Post Combined License Activities

There are no post-COL activities related to this subsection.

2.4S.5.6 Conclusion

The staff reviewed the application and confirmed that the applicant has addressed the required information related to estimating the flood levels caused by a hurricane storm surge from the Gulf of Mexico, and no outstanding information is required to be addressed in the COL FSAR related to this section. The staff determined that the applicant's site-specific PMSS maximum water surface elevation of 8.9 m (29.3 ft) MSL is reasonable and conservative.

As set forth above, the applicant presents and substantiates information to establish the site description. The staff reviewed the applicant's information and for the reasons stated above, the staff concluded that as documented in Section 2.4S.5, of this SER, the applicant has provided sufficient details about the site description to allow the staff to evaluate whether the applicant has met the relevant requirements of 10 CFR Part 52.79(a)(1) and 10 CFR Part 100, with respect to determining the acceptability of the site. The information addressing COL Information Items 2.4 and 3.5 is therefore accurate and acceptable.

2.4S.5.7 References

Blain, C.A., J.J. Westerink, and R.A. Luettich, "Domain and grid sensitivity studies for hurricane storm surge predictions," Computational Methods in Water Resources X, A. Peters et al., (eds), Heidelberg, July, 1994.

Bodine, B.R., "Storm Surge on the Open Coast: Fundamentals and Simplified Prediction," Technical Memorandum No. 35, U.S. Army Corps of Engineers Coastal Engineering Research Center, May 1971.

Bunya S, J. Westerink, J.C. Dietrich, H.J. Westerink, L.G. Westerink, J. Atkinson, B. Ebersole, J.M. Smith, D. Resio, R. Jensen, M.A. Cialone, R. Luettich, C. Dawson, H.J. Roberts, and J. Ratcliff, "A High Resolution Coupled Riverine Flow, Tide, Wind, Wind Wave and Storm Surge Model for Southern Louisiana and Mississippi: Part I–Model Development and Validation," *Monthly Weather Review* 138:345–377, 2010.

Demirbilek, Z., L. Lin, and D. J., Mark, "Numerical Modeling of Storm Surges in Chesapeake Bay," *International Journal of Ecology & Development*, Special Issue on Coastal Environment, 10(S08), 2008.

Dietsche, D., S.C. Hagen, and P. Bacopoulos, "Storm Surge Simulations for Hurricane Hugo (1989): On the Significance of Inundation Areas," *ASCE Journal of Waterways, Port, Coastal, and Ocean Engineering*, 133(3), pp. 183–191, 2007.

Federal Emergency Management Agency, 2010, "Numerical Models Meeting the Minimum Requirement of National Flood Insurance Program," available at http://www.fema.gov/plan/prevent/fhm/en_coast.shtm , accessed December 20, 2010.

Funakoshi, Y., S. C. Hagen, and P. Bacopoulos, "Coupling of hydrodynamic and wave models: Case study for Hurricane Floyd (1999) hindcast," *Journal of Waterway, Port, Coastal, and Ocean Engineering*, 134(6), pp. 321–335, 2009.

Gibeaut, J. C., et al.,, "Texas Shoreline Change Project: Gulf of Mexico Shoreline Change from the Brazos River to Pass Cavallo," Texas Bureau of Economic Geology, October 2000.

Gica, E., M. H. Teng, R. A. Luettich, K. F. Cheung, C. A. Blain, C-S Wu, and N. W. Scheffner, "Numerical Modeling of Storm Surge Generated by Hurricane Iniki in Hawaii," in WAVES 2001, Proceedings of the 4th International Symposium on Ocean Wave Measurement and Analysis, American Society of Civil Engineers, Vol. 2, pp. 1555–1564, September 2–6, 2001, San Francisco, California, 2001.

Grenier, R.R., R.A. Luettich, and J.J. Westerink, "Comparison of 2D and 3D models for computing shallow water tides in a friction dominated tidal embayment," Estuarine and Coastal Modeling III, M. Spaulding et al.,, (eds), American Society of Civil Engineers, New York, NY, pp. 58–70, 1994.

Halff Associates, Inc., "1992 Colorado River Flood Damage Evaluation Project, Final Report, Phase I, Volume I and Volume II," prepared for the Lower Colorado River Authority and Fort Worth District Corps of Engineers, July 2002.

Holland G.J., "An analytic model of the wind and pressure profiles in hurricanes," *Monthly Weather Review* 108:1212–1218, 1980.

Jelesnianski, C.P., J. Chen, and W. A. Shaffer, "SLOSH: Sea, Lake, and Overland Surges from Hurricanes," National Oceanic and Atmospheric Administration, Technical Report NWS 48, National Weather Service, Silver Spring, MD, April 1992.

Luettich, R.A., Jr., J.L. Hench, C.D. Williams, B.O. Blanton, and F.E. Werner, "Tidal circulation and larval transport through a barrier island inlet," Estuarine and Coastal Modeling V, M. Spaulding et al., (eds), American Society of Civil Engineers, pp. 849–863, 1998.

Luettich, R.A. and J.J. Westerink, “A three dimensional circulation model using a direct stress solution over the vertical,” Computational Methods in Water Resources IX, Volume 2: Mathematical Modeling in Water Resources, T. Russell et al., (eds), Computational Mechanics Publications, Southamption, UK, 1992.

Luettich, R.A., Jr., S. Hu, J.J. Westerink, and N.W. Scheffner, “Modeling 3-D Circulation Using Computations for the Western North Atlantic and Gulf of Mexico,” Estuarine and Coastal Modeling II, M. Spaulding (ed.), American Society of Civil Engineers, pp. 632–643, 1992.

National Oceanic and Atmospheric Administration, “Meteorological Criteria for Standard Project Hurricane and Probable Maximum Hurricane Windfields, Gulf and East Coasts of the United States,” NOAA Technical Report NWS 23, U.S. Department of Commerce, Washington, DC, September 1979.

National Oceanic and Atmospheric Administration, “Mean Sea Level Trend 8772440 Freeport, Texas,” 2009; http://tidesandcurrents.noaa.gov/sltrends/sltrends_station.shtml?stnid=8772440%20Freeport,%20TX ; accessed April 3, 2009.

Resio, D., “Modern Methods for Coastal Storm Surge Estimation,” presentation at NRC Hydrology Research Workshop in Rockville, MD, U.S. Army Corps of Engineers, Coastal and Hydraulics Laboratory, Engineer Research and Development Center, Vicksburg, MS, November 17–18, 2009 (ML102990221).

South Texas Nuclear Operating Company, “South Texas Project Combined License Application, Revision 0, Part 2, Final Safety Analysis Report,” 2007.

U.S. Army Corps of Engineers, “Final Environmental Assessment, Emergency Repairs to Texas City and Vicinity, Texas Hurricane Protection Project, Galveston County, Texas,” Galveston District, April 2009.

U.S. Army Corps of Engineers, “Coastal Engineering Manual – Part II,” Engineering Manual 1110-2-1100, Coastal and Hydraulics Laboratory, Engineer Research and Development Center, Vicksburg, MS, 2008.

Vickery, P.J., “Hurricane Hazard Modeling,” presentation at NRC Hydrology Research Workshop in Rockville, MD, Applied Research Associates, Inc., Raleigh, NC, November 17-18, 2009 (ML102990157).

Westerink, J.J., R.A. Luettich, and J.C. Muccino, “Modeling tides in the Western North Atlantic using unstructured graded grids,” Tellus, 46A, pp. 178–199, 1994.

Westerink, J.J., R.A. Luettich, A.M. Baptista, N.W. Scheffner and P. Farrar, “Tide and storm surge predictions using finite element model,” *ASCE Journal of Hydraulic Engineering*, 118(10), pp. 1373–1390, 1992.

2.4S.6 Probable Maximum Tsunami

2.4S.6.1 Introduction

This section of the FSAR addresses the hydrological design basis developed to ensure that any potential tsunami hazards to the SSCs important to safety are considered in the plant's design.

This SER section presents the staff's review of the flood levels caused by postulated tsunami scenarios. The specific areas of the review include the description of the PMT, historical tsunami records, source generator characteristics, tsunami analyses, tsunami water levels, hydrography and harbor or breakwater influences on a tsunami, and the effects on safety-related facilities.

2.4S.6.2 Summary of Application

In Section 2.4S.6, the applicant provides site-specific information about potential tsunami effects on the site. In addition, in FSAR Section 2.4S.4, the applicant provides site-specific information to address COL License Information Items 2.14 and 3.5:

COL License Information Items

- COL License Information Item 2.14 Floods

COL License Information Item 2.14 requires COL applicants to provide site-specific information related to historical flooding and the potential flooding at the plant site including flood history, flood design considerations, and effects of local intense precipitation. This information is provided below.

In FSAR Section 2.4S.6, the applicant evaluates several different tsunami sources from the published scientific literature to establish the PMT at the site. Approximate tsunami wave heights are indicated by Knight (2006) for four seismogenic sources located in the Caribbean and the Gulf of Mexico and by Mader (2001) for the 1755 Lisbon earthquake, which was located in the Atlantic Ocean. The wave height estimate from Trabant et al., (2001) for the East Breaks submarine landslide is considered highly unlikely by the applicant.

After reviewing published tsunami catalogs, databases, and historical accounts, the applicant identifies the following three historical tsunami events for the STP site:

- An October 11, 1918, seismogenic tsunami originating west of Puerto Rico.
- A May 2, 1922, seismogenic tsunami originating near the Virgin Islands.
- A March 27, 1964, Gulf of Alaska earthquake generating seismic seiche waves (not a tsunami event in the Gulf of Mexico)

The applicant examines published information to determine the source generator characteristics for several different types of potential tsunami sources: seismogenic, volcanogenic, and landslide generated. For seismogenic tsunamis, the applicant discusses the propagation characteristics into the Gulf of Mexico for earthquakes located in the Caribbean and the Atlantic Ocean (Knight, 2006). For volcanogenic tsunamis (catastrophic flank failures), the applicant cites recent studies to discount the La Palma, Canary Islands transoceanic tsunami scenario published by Ward and Day (2001). For landslide-generated tsunamis, the applicant discounts the East Breaks landslide tsunami scenario published by Trabant et al., (2001) as highly unlikely.

To determine the maximum tsunami water levels, the applicant uses an estimate of the tsunami in the Gulf of Mexico from a near-field submarine landslide near the East Break slump and then applies (1) a runup amplification factor, (2) 10 percent exceedance of an astronomical high tide according to RG 1.59 (1977), and (3) a sea level rise from global climate change in the next century. The applicant determines the maximum water level for the PMT at 11.5 feet above MSL.

Therefore, the applicant concludes that the flood elevation at STP Units 3 and 4 due to the postulated PMT event will not be the controlling design-based flood elevation for STP Units 3 and 4 because it is below the plant grade, and there will be no onsite effects from tsunami-breaking waves or resonance or onsite tsunami waves on safety-related facilities.

2.4S.6.3 Regulatory Basis

The associated acceptance criteria for the PMT hazards are in Section 2.4.6 of NUREG-0800.

The regulatory requirements that establish the acceptance criteria for reviewing this section are as follows:

- 10 CFR Part 50 Appendix A, GDC 2, requires the COL applicants to consider the most severe of the natural phenomena that have been historically reported for the site and surrounding area, with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

- 10 CFR 52.79(a)(1)(iii), requires the COL applicants to identify hydrologic site characteristics with appropriate consideration for the most severe of the natural phenomena that have been historically reported for the site and surrounding areas, with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

- 10 CFR 100.20 specifies the factors to be considered when evaluating sites. 10 CFR 100.20(c) specifies the requirements for considering the physical characteristics of a site (including seismology, meteorology, geology, and hydrology) to determine its acceptability to host a nuclear unit(s).

- 10 CFR 100.23(d)(3) sets forth the criteria to determine the siting factors for plant design bases with respect to seismically induced floods and water waves at the site. Section IV(c) of Appendix A to Part 100 specifies the required information for seismically induced floods and water waves, including distantly and locally generated tsunami runup and drawdown, local coastal topography that affects the tsunami runup and drawdown, geologic and seismic evidence for evaluating seismically induced floods and water waves, and probable slip characteristics of offshore or nearby lakes and rivers.

In addition, the staff used the regulatory positions of the following regulatory guides for the identified acceptance criteria:

- RG 1.27 describes the applicable UHS capabilities.
- RG 1.59, as supplemented by the best current practices, provides guidance for developing the design flood bases.

2.4S.6.4 Technical Evaluation

NRC staff reviewed the information in Section 2.4S.6 of the STP Units 3 and 4 COL FSAR. The staff's review confirmed that the information in the application addresses the relevant information related to the PMT. The staff's technical review of this section included an independent review of the applicant's information in the FSAR and in the responses to the RAIs. The staff supplemented this information with other publicly available sources of data.

This section describes the staff's evaluation of the technical information in FSAR Section 2.4S.6.

COL License Information Items

- COL License Information Item 2.14 Floods

The staff reviewed the applicant's supplemental information on tsunami-generated floods. The staff's review of the application is summarized below:

2.4S.6.4.1 Probable Maximum Tsunami

Information Submitted by Applicant

The applicant evaluates several different tsunami sources from the published scientific literature to establish the PMT. Approximate tsunami wave heights are indicated for four seismogenic sources located in the Caribbean and the Gulf of Mexico and for the 1755 Lisbon earthquake, which was located in the Atlantic Ocean. In the FSAR, the applicant states that the wave height estimate for the East Breaks submarine landslide is highly unlikely. However, the applicant revises the potential for tsunamis from the East Breaks landslide in the response to **RAI 02.04.06-1** dated December 4, 2008 ML083460084) and in the FSAR.

RAI 02.04.06-1 requests the following information:

> [Item 1] Provide a tsunami modeling analysis of the East Breaks landslide to clarify whether the 7.6-m (24.93-ft) offshore wave height indicated by Trabant et al., (2001) can be discounted.
>
> [Item 2] In addition, provide additional tsunami analyses of other regions in the Gulf of Mexico that are prone to landslides.
>
> [Item 3] To independently validate whether a tsunami hazard exists for the proposed site, provide geologic methods and tsunami identification criteria used to justify the determination that no tsunami deposit was found at the site.
>
> [Item 4] Provide excavation photos from Units 1 and 2.

[Item 5] Indicate if there are geologically conducive locations for the deposition and preservation of tsunami deposits at the STP site or nearby regions.

NRC Staff's Technical Evaluation

Resolution of the significant items [Items 1 and 2] of **RAI 02.04.06-1** is discussed below.

[Item 1] East Breaks Landslide: The applicant's response to the RAI, dated December 4, 2008 (ML083460084), provides the geologic background and four possible source scenarios for landslide tsunamis in the East Breaks region. The geologic background for the East Breaks landslide is taken primarily from published literature and, in general, presents a reasonable summary. The applicant also provides the theoretical basis of the tsunami propagation used (Method of Splitting a Tsunami [MOST]) and its verification. However, the applicant does not thoroughly discuss the conservatism of input parameters. The applicant uses a large (but physically reasonable) bottom-roughness coefficient (i.e., 0.01 on page 4 of the response) that may not give the most conservative estimate of the water level. The generation phase of the applicant's simulations is based on a slump center-of-mass motion model, in which the time history of slide movement is specified only for the center of the mass of a slide with a prescribed geometry (e.g., Gaussian shape). This model contrasts with using the full-time varying displacement field for submarine mass failures as initial conditions for tsunami generation. The center-of-mass motion model may be adequate during the early stages of a post-failure slide movement but does not account for changes in deformation, as the landslide fully mobilizes down the slope. The staff determined that the response to RAI 02.04.06-1 Item 1 is acceptable.

[Item 2] Other Gulf of Mexico Landslides: The applicant provides a descriptive justification for why other Gulf of Mexico landslide provinces are not considered in establishing the PMT for the site. These provinces are the Mississippi Canyon, Florida Escarpment, and Campeche Escarpment (ten Brink et al., 2008). The applicant maintains that there is a significant diffusion and energy dissipation associated with landslides that are more distant than the East Breaks landslide. It is unclear whether the applicant performed an additional tsunami analysis for the more distant landslides to make this conjecture.

In the FSAR the applicant concluded that the more distant landslides in the Gulf of Mexico with propagation paths oblique to the site are not likely to have potential runup heights greater than those from the East Breaks Landslide. However, the applicant does not provide sufficient justification for dismissing the possibility that the Campeche Escarpment region may be a potential source region that determines the PMT water levels. To evaluate the potential tsunami effects of these submarine landslide sources, the staff performed an independent confirmatory analysis that estimated the PMT water levels.

Confirmatory analysis and major findings: A detailed description of the NRC staff's independent confirmatory analysis to determine the PMT at the STP site is in the sections that follow. In summary, the staff considered both far-field seismogenic and near-field (Gulf of Mexico) landslide sources as potential generators for the PMT. An initial analysis indicates that submarine landslides broadside (i.e., directly across) from the site are the likely sources that determine the PMT (See SER Subsection 2.4S.6.4.3). This analysis includes the East Breaks landslide and potential landslides along the Campeche Escarpment. Each landslide source has a unique hydrodynamic behavior described below in Subsection 2.4S.6.4.5. Within the uncertainty of the tsunamigenic source data, either could be the PMT source.

Conclusion: The applicant's response to **RAI 02.04.06-1** and the NRC staff's confirmatory analysis differ significantly in the descriptions of how to determine the PMT. However, the applicant's PMT water level estimate (3.5 m [11.5 ft] MSL) represents a near-shore/coastal location that is less than the staff's PMT water level estimate of 5 m (16.4 ft) MSL for an inland location closer to the STP site, taking into account the effect of an overland flow. Moreover, the PMT surge level estimates by both the applicant and the staff are far below the bounding main cooling reservoir breach water level of 12.2 m (40.0 ft) MSL or the plant grade of 10.36 m (34 ft) MSL. Thus, the staff concluded that the postulated PMT would not affect the proposed STP site. Therefore RAI 02.04.06-1 is closed.

2.4S.6.4.2 Historical Tsunami Record

Information Submitted by Applicant

After reviewing published tsunami catalogs, databases (such as National Geodetic Data Center), and historical accounts, the applicant identifies three historical tsunami events for the STP site. These include (1) an October 11, 1918, seismogenic tsunami originating west of Puerto Rico; (2) a May 2, 1922, seismogenic tsunami originating near the Virgin Islands; and (3) seismic seiche waves originating from the March 27, 1964, Gulf of Alaska earthquake (not a tsunami event in the Gulf of Mexico).

NRC Staff's Technical Evaluation

NRC staff conducted a review of this historical record to confirm whether the three events listed by the applicant are the primary tsunamis and seismic seiches measured and observed along the Gulf Coast. An additional entry in the National Geodetic Data Center (NGDC) tsunami database for the Gulf of Mexico is an event that occurred at Grand Isle, Louisiana, on September 22, 1909. As indicated in the database, this event was likely caused by a hurricane, not by a tsunami. See Geist et al., (2009) for a discussion of other historic tsunamis.

The applicant does not address possible evidence for paleotsunami deposits in the FSAR Section 2.4S.6. For example, a deposit located north of the site in Falls County, Texas, near the Brazos River was originally interpreted by Bourgeois et al., (1988) as caused by a paleotsunami. The Brazos deposit is dated at or near the time of the Cretaceous-Tertiary boundary and is located at the paleo-shoreline for that time period. Since that time, the Gulf Coast shoreline has transgressed southward to its current geographic position. The common interpretation of this deposit is that owing to its date and the existence of impact ejecta, it was emplaced by a tsunami generated from a Chicxulub asteroid impact at the Brazos site. Bourgeois et al., (1988) suggested that a tsunami wave 50 to 100 m (164 to 328 ft) high was necessary to explain this deposit. It is not conceivable that the wave that created these deposits was generated by any landslide source that would be of relevance to the present day PMT determination. It is likely that a wave of the estimated height would be caused by a relatively nearby large impact event. Waves emanating from such a source would have the extreme wave heights and long periods needed to be able to propagate significant wave energy this far inland.

Conclusion: NRC staff examined primary references for historical observations and measurements of tsunami and seismic seiche waves occurring along the Gulf Coast. Except for the date of the 1918 hydrologic event and the source for the 1922 hydrologic event, the staff's assessment of the historical record is consistent with that of the applicant's. Additionally, the applicant does not consider the existence of a possible paleotsunami that occurred along the ancient Gulf Coast shoreline, currently located along the Brazos River in Falls County, Texas. The common interpretation of this deposit is that it was emplaced by a tsunami generated by the Chicxulub impact. It is unlikely, however, that the wave heights inferred from the deposit are relevant to a determination of the present day PMT. Therefore, the staff finds the applicant's analysis acceptable.

Source Generator Characteristics

Information Submitted by Applicant

In FSAR Subsection 2.4S.6.3, the applicant states that it examined published information to determine the source generator characteristics for several different types of tsunamis: seismogenic, volcanogenic, and landslide generated. For seismogenic tsunamis, the applicant discusses the propagation characteristics into the Gulf of Mexico for earthquakes located in the Caribbean and the Atlantic Ocean. For volcanogenic tsunamis, the applicant cites recent studies to discount the La Palma, Canary Islands transoceanic tsunami scenario. For landslide-generated tsunamis, the applicant discounts the East Breaks landslide tsunami scenario published by Trabant et al., (2001) as highly unlikely, though the applicant revisits this scenario in the response to **RAI 02.04.06-1**.

NRC Staff's Technical Evaluation

This section describes the tsunami sources used for the independent confirmatory analysis, including parameters associated with the maximum submarine landslides in the Gulf of Mexico. The end of this section includes a brief discussion of seismic seiches.

Potential tsunami sources that are likely to determine the PMT at the STP site are submarine landslides in the Gulf of Mexico. Subaerial landslides, volcanogenic sources, near-field intra-plate earthquakes and inter-plate earthquakes along the Caribbean plate boundary faults are unlikely to be the causative tsunami generator for the PMT at the STP site.

Subaerial Landslides: With regard to subaerial landslides, there are no major coastal cliffs near the site that would produce tsunami-like waves that exceed the amplitude of those generated by other sources.

Volcanogenic Sources: According to the Global Volcanism Program of the Smithsonian Institution (http://www.volcano.si.edu/), there are three general regions of volcanic activity that have the potential to generate localized wave activity in the Gulf of Mexico and the Caribbean Sea: (1) two Mexican volcanoes near the Gulf of Mexico coastline, (2) two volcanoes in the western Caribbean, and (3) volcanic activity along the Lesser Antilles island arc. Catastrophic failures associated with volcanoes along the eastern coasts of Mexico and Central America is either too far inland or too small in size to generate significant wave activity near the STP site. Based on existing evidence, volcanoes along the Lesser Antilles or in the eastern Atlantic Ocean are too far away to generate significant wave activity in the Gulf of Mexico.

Intra-Plate Earthquakes: Because there are no tectonic plate boundaries in the Gulf of Mexico region, earthquakes *local* to the STP site occur in an intra-plate tectonic environment, thus limiting the maximum magnitude these earthquakes can attain (M_{max} = 7.5; see Petersen et al., 2008, for details of this analysis). Because the maximum slip, and consequently the maximum sea floor displacement, associated with an earthquake scale with its magnitude, the initial tsunami wave amplitude associated with an intra-plate earthquake would therefore be less than that used for local submarine landslides under conservative conditions, as described below in Subsection 2.4S.6.4.5.

Inter-Plate Earthquakes: In the far-field description of major plate boundary faults, Chapter 8 of ten Brink et al., (2008) estimates specific source parameters and offshore tsunami amplitudes of Caribbean inter-plate earthquakes. The tsunami propagation model in ten Brink et al., (2008) was refined during the NRC staff's confirmatory analysis for two of the principal faults (the northern South American Convergent Zone and the northern Caribbean Subduction Zone) using the Cornell Multigrid Coupled Tsunami Model (COMCOT) (See Subsection 2.4S.6.4.5 below).

Local Submarine Landslides: Submarine landslides in the Gulf of Mexico are considered a potential tsunami hazard for the STP site for two reasons: (1) some dated landslides in the Gulf of Mexico have post-glacial ages, suggesting that the triggering conditions for these landslides are still present; and (2) analyses of recent seismicity suggest the presence of small-scale energetic landslides in the Gulf of Mexico. NRC staff defined four geological provinces in the Gulf of Mexico that are likely to be the origin of submarine landslides that control the determination of the PMT: northwest of the Gulf of Mexico (immediately off the STP site), the Mississippi Canyon, the Florida Escarpment, and the Campeche Escarpment. The first is a mixed canyon/fan and salt province involving the failure of terrigenous and hemipelagic sediment; the second is a canyon/fan province; and the third and fourth are carbonate provinces formed from reef structures and characterized by steep slopes (i.e., escarpments).

Because the Mississippi Canyon and Florida Escarpment landslides are oblique to the STP site, the length of the continental shelf that the wave must travel over is much greater than that for the East Breaks landslide or for landslides along the Campeche Escarpment that are broadside from the STP site. This would result in much greater energy dissipation during propagation that is associated with tsunamis from the Mississippi Canyon and Florida Escarpment source regions. The characteristics and the parameters that define the maximum landslide are given below.

The primary landslide parameters that are used in the tsunami models include the excavation depth and the slide width, which can be directly measured from sea floor mapping of the largest observed slide in the four geologic provinces. The other necessary parameter is the downslope landslide length, which is interpreted from the runout distance. The runout distance measured from sea floor mapping is a combination of fast plug flow (low viscosity, non-turbulent); creeping plug flow (high viscosity/viscoplastic; non-turbulent); and turbidity currents (turbulent boundary layer fluid). The latter two likely have little to no tsunami-generating potential. The landslide lengths indicated below are intended to represent the main tsunami-generating phase. The amplitude of the initial negative wave above the excavation region is linked to the maximum excavation depth. The amplitude of the initial positive wave above the deposition region is determined from a conservation of landslide volume. The excavation volume can be determined using GIS techniques (see below). Setting the deposition volume equal to the excavation volume determines the positive amplitude for a given landslide length. For a fixed volume, increasing the landslide length decreases the initial positive amplitude of the tsunami.

Landslide volume calculations are based on measuring the volume of material excavated from the landslide source area using a technique similar to that of ten Brink et al., (2006) and Chaytor et al., (2009). Briefly stated, the approach involves using multibeam bathymetry to outline the extent of the excavation area, interpolating a smooth surface through the polygons that define the edges of the slide to provide an estimate of the pre-slide slope surface, and subtracting this surface from the present seafloor surface.

The maximum observed landslide from multibeam surveys is taken as the maximum landslide for a given region. It may be possible that larger landslides could occur in a given region. However, this determination of the maximum landslide is consistent with the overall definition of the PMT as "the most severe of the natural phenomena that have been historically reported or determined from geological and physical data for the site and surrounding area." In this case, the maximum landslide is taken from geologic observations spanning tens of thousands of years.

a. *East Breaks Landslide*

Geologic Setting: The river delta that formed at the shelf edge during the early Holocene.

Age: 10,000 to 25,000 years.

Maximum Single Event: Maximum and minimum parameters are taken from different interpretations of the digitized failure scar surrounding the excavation region (Chaytor et al., 2009).

Volume	Area	Width	Length	Excavation Depth	Runout Distance
Max: 21.95 km^3 Min: 20.80 km^3	519.52 km^2 420.98 km^2	~ 12 km	~ 50 km	~160 m	91 km*
*From the toe of the excavation area and 130 km from the headwall based on GLORIA data. Note that the multibeam bathymetry is not available for the entire runout area.					

b. *Mississippi Canyon*

Geologic Setting: River delta and fan system.

Age: 7,500 to 11,000 years.

Maximum Single Event

Volume	Area	Excavation Depth	Runout Distance
425.54 km^3	3687.26 km^2	~300 m	297 km*
*From the toe of the excavation area and 442 km from the headwall scarp.			

c. *Florida Escarpment*

Geologic Setting: Edge of a carbonate platform.

Age: Early Holocene or older. Because the Florida Escarpment carbonate failures are buried by Mississippi Fan deposits, the Florida Escarpment failures are older than the youngest fan deposits dated at about 11,500 years old.

Maximum Single Event

Volume	Area	Excavation Depth	Runout Distance
16.2 km^3	647.57 km^2	~150 m, but quite variable	Uncertain*
*The landslide deposit is at the base of the Florida Escarpment and is buried under younger Mississippi Fan deposits.			

d. *Campeche Escarpment*

Geologic Setting: Carbonate platform.

Age: No specific data are available

Maximum Event: *No specific data is available nor obtainable because the East Break is located within the territory of Mexico.* One of the persistent issues during the independent confirmatory analysis is acquiring sufficient geologic information about the Campeche Escarpment with which to estimate the maximum landslide parameters, as with the other Gulf of Mexico landslide provinces. Plans to conduct multibeam bathymetry surveys are pending. Presently, there is no published information showing the detailed bathymetry or distribution of landslides on or above the Campeche Escarpment.

Seismic Seiches

Rather than being impulsively generated by the displacement of the sea floor, seismic seiches occur from the resonance of seismic surface waves within enclosed or semi-enclosed bodies of water. The harmonic periods of the oscillation are dependent on the dimensions and geometry of the body of water. In 1964, seiches were set up along the Gulf Coast from seismic surface waves emanating from the M = 9.2 Gulf of Alaska earthquake, owing (in part) to the amplification of seismic waves from the thick sedimentary section along the Gulf Coast. Because the propagation path from Alaska to the Gulf Coast is almost completely continental, and because the magnitude of the 1964 earthquake is close to the maximum possible for that subduction zone, it is likely that the historical observations of the 1964 seiche wave heights are the maximum possible and less than the PMT amplitudes from landslide sources.

In summary, the discussion that follows is a list of the findings in the NRC staff's independent confirmatory analysis of the tsunami source characteristics:

- There is sufficient evidence to consider submarine landslides in the Gulf of Mexico a present day tsunami hazard for the purpose of defining the PMT at the STP site.
- Four geologic landslide provinces are defined in the Gulf of Mexico that are applicable for determining the PMT: northwest of the Gulf of Mexico, the Mississippi Canyon, the Florida

Escarpment, and the Campeche Escarpment. The propagation paths that result in the least attenuation of potential tsunamis are the East Breaks and the Campeche provinces.

- Parameters for the maximum submarine landslide were determined for each of the provinces, except for the Campeche Escarpment (which is awaiting additional data).

- It is likely that seismic seiche waves resulting from the 1964 Gulf of Alaska earthquake are nearly the highest possible owing to a predominantly continental ray path for seismic surface waves from Alaska to the Gulf Coast.

2.4S.6.4.3 Tsunami Analysis

Information Submitted by Applicant

Based on the review of tsunami sources, the applicant indicates that modeling tsunami wave heights and periods at the site is not warranted and was not performed. However, the applicant conducted a tsunami analysis in response to **RAI 02.04.06-1**.

NRC Staff's Technical Evaluation

The most common computational models include MOST, COMCOT, and TSUNAMI2. All three models solve the same depth-integrated and 2D-horizontal (2DH) nonlinear shallow-water (NSW) equations with different finite-difference algorithms. There are a number of other tsunami models, including the finite element model ADCIRC.

Earthquake-generated tsunamis, with their very long wavelengths, are ideally matched with NSW equations for transoceanic propagation. Models such as MOST and COMCOT have been shown to be reasonably accurate throughout the evolution of a tsunami and are in widespread use today. However, when examining the tsunamis generated by submarine mass failures, the NSW equations can lead to significant errors (Lynett et al., 2003). The length scale of a submarine failure tends to be much less than that of an earthquake, and thus the wavelength of the created tsunami is shorter. To correctly simulate the shorter wave phenomenon, there needs to be equations with excellent shallow to intermediate water properties, such as the Boussinesq equations. Thus, for the work proposed here, the Boussinesq-based numerical model COULWAVE (Lynett and Liu, 2002) will be used. For technical details on wave propagation, breaking, runup, inundation, and overtopping of sloping structures see Geist et al., (2009) (including the references).

In response to **RAI 02.04.06-1**, the applicant models a tsunami from the East Breaks landslide using a NSW wave model (MOST) that is described in FSAR Version 3.0. In contrast, NRC staff used a higher-order Boussinesq hydrodynamics model (COULWAVE) in the staff's confirmatory analysis. This model is more specifically suited to landslide tsunamis.

2.4S.6.4.4 Tsunami Water Levels

Information Submitted by Applicant

To determine the maximum tsunami water levels, the applicant used a published estimate of the tsunami in the Gulf of Mexico from a near-field submarine landslide near the East Break slump and then applied (1) a runup amplification factor, (2) 10 percent exceedance of an astronomical high tide, and (3) sea level rise from global climate change. The applicant's finding for the PMT maximum water level is 11.5 feet above MSL, which includes the effects of the high tide exceedance and sea level rise in the next century on the site.

NRC Staff's Technical Evaluation

An independent confirmatory analysis of tsunami water levels at the STP site focuses on distant earthquake tsunami sources and landslide sources local to the Gulf of Mexico.

a. Distant Earthquake Sources

For comparative purposes, NRC staff re-computed the offshore tsunami water levels for the northern Caribbean subduction zone and the northern South American convergent zone earthquake scenarios of ten Brink et al., (2008). These scenarios use the COMCOT model that includes non-linear terms and a moving boundary condition at the shoreline and computes the model in spherical coordinates. Bottom friction is also included but is set at a low, conservative value ($f = 10^{-4}$) in this case. These results confirm that tsunami amplitudes from distant Caribbean earthquakes are less than 1.0 m (3.28 ft) near the STP site. Tsunami amplitudes from earthquakes along the Azores-Gibraltar oceanic convergence boundary are also likely to be less than 1 m (3.28 ft) in the Gulf of Mexico. Therefore the staff finds the applicant's analysis acceptable.

b. Local Landslide Sources

A detailed tsunami analysis was performed for two local landslide scenarios: (1) the East Breaks landslide, and (2) a hypothetical landslide along the Campeche Escarpment. For each case, COULWAVE was used to compute the tsunami propagation, runup, and inundation (see Subsection 2.4S.6.4.4). For the development of the numerical grid and for additional details, see Geist et al., (2009). Therefore the staff finds the applicant's analysis acceptable.

Initial Numerical Simulations – Physical Limits

The purpose of these initial staff simulations are to provide an upper limit of the tsunami wave height that could be generated by the Gulf of Mexico landslide scenario. Source parameters for the simulation include landslide width, length, and excavation depth. Although the landslide volume is not a direct parameter that was used in the model, the volumes of excavation and deposition were conserved and used to determine the amplitude of the initial positive wave. Note that these limiting simulations used physical assumptions that are arguably unreasonable. The results of these simulations will be used to filter out tsunami sources that are incapable of adversely impacting the STP site under even the most conservative assumptions. Specifically, these staff assumptions are:

1. The time scale of the submarine landslide motion is very small (i.e., instantaneous) compared to the period of the generated tsunami.

2. Bottom roughness and the associated energy dissipation are negligible in locations that are initially wet (i.e., locations with a negative bottom elevation offshore).

With Assumption 1, the free-water surface response matches the change in the seafloor profile exactly. The landslide time-evolution parameter, which is associated with a high degree of uncertainty, is thus removed. Assumption 2 prevents the use of an overly high bottom-roughness coefficient, which could artificially reduce the tsunami energy reaching the shoreline. Such an assumption is too physically unrealistic to accept for the inland regions, where the roughness height may be the same order as the flow depth. For tsunami inundation, particularly for regions similar to the location of this project where the wave would need to inundate long reaches of densely vegetated land to reach the site, it is necessary to include a conservative measure of bottom roughness.

East Breaks Landslide

1HD Results: For the East Breaks landslide, both 1 and 2 horizontal-dimension (HD) simulations were performed. The 1HD simulations do not include radial spreading (representing the extreme case of an infinitely wide landslide) and refraction effects. Refraction can have either a constructive or destructive effect on the wave height, depending on the shallow water depth contours.

Three 1HD simulations were performed for cases of varying on-shore bottom friction: (a) bottom friction due to the small roughness characteristic of a very smooth and sandy ground (bottom-drag coefficient, $f = 0.001$); (b) bottom friction due to the small/moderate roughness characteristic of grass/turf ($f = 0.01$); and (c) bottom friction due to the large roughness characteristic of the trees and the dense, shrub-like vegetation that currently exists seaward of the STP reservoir ($f = 0.05$).

The Low Friction Case "a" shows a fast-moving bore front that easily overtops the STP main cooling reservoir, with maximum water surface elevations approaching about 98 ft (30 m). Despite the relatively low friction value used in Case "b," the tsunami wave front slows significantly here. The wave does not overtop the main cooling reservoir, and maximum water elevations near the STP site are approximately 33 ft (10 m) (Figure 2.4S.6.4.5). Finally, for Case "c," the large realistic friction retards the flow considerably, and the tsunami wave front does not reach the STP site but still manages to travel 10 km (6.22 mi) inland. A conclusion of this 1HD East Breaks study is that a tsunami approaching the site, with a bore height up to about 30 m (98 ft) at the still water shoreline, will not adversely impact the site if the vegetation roughness is properly accounted for. Again, the 1HD case does not include the lateral dissipation (radial spreading) of the wave from the source.

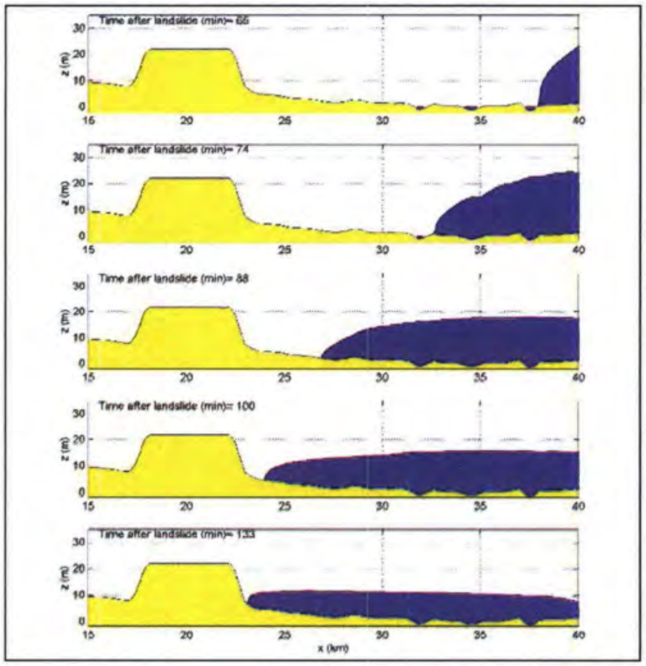

Figure 2.4S.6.4.5-1 The Onshore Evolution of the 1HD Tsunami from the East Breaks Scenario for the Mid-Friction Case (Case "b"). A Cross-Sectional Profile of the Main Cooling Reservoir is Shown on the Left Side.

2HD Results: The 2HD simulation provides information about the importance of radial spreading and refraction, which can be used to qualitatively correct the 1HD results. With no refractive amplification and significant radial spreading, the 2HD tsunami height is less than the 1HD near the shoreline, with the 2HD simulation yielding bore height predictions on the order of about 10 m (33 ft) at the shoreline, or 1/3 of the 1HD prediction. Considering this 2HD spreading reduction with the 1HD inundation results and the conservative "hot-start" approach that the simulation employed, it can be stated with high certainty that the tsunami from the East Breaks landslide will not impact the STP site.

Uncertainty in the primary landslide source parameters for the tsunami (excavation depth and slide length) is, to a great extent, diminished owing to the depth-limiting effects on the amplitude during propagation across the south Texas continental shelf. Depth-limiting effects mean that for a given beach profile and incident wave period, there is some ratio of wave height to shelf water depth that remains more or less constant, as the wave propagates across the broad continental shelf.

Campeche Landslide

Presently, there is no available information showing the detailed bathymetry or the distribution of landslides on or above the Campeche Escarpment. As a provisional source for the Campeche Escarpment, NRC staff used initial conditions applicable to the maximum observed landslide along the Florida Escarpment (a similar geologic environment). The Campeche Escarpment is includes an initial drawdown of 150 m (492 ft), with a horizontal length scale of 20 km (12.43 mi). The very steep slope of the Campeche Escarpment results in the maximum depression occurring over a depth of 500 m (1,640 ft), whereas the maximum positive wave of the initial condition occurs over a depth of 1,000 m (3,281 ft). Because the propagation distance for Campeche is much larger than that of East Breaks (about 700 km [435 mi] longer), the two-dimensional spreading effect will likely be very significant and will result in a greater attenuation than for the East Breaks scenario. For the 2HD setup, slide widths of 20 km and 60 km (12.43 mi and 31.08 mi) were tested. The former is the expected maximum for the Florida Escarpment; the latter is similar to the maximum width in the Storegga landslide complex and the width for the "Monster" scenario landslide that the applicant used for the south Texas continental shelf. In both cases, the wave heights decrease very quickly near the source, but they reach a nearly steady (slowly attenuating) condition when they reach the continental shelf off the Gulf Coast. SER Figure 2.4S.6.4.5-2 shows a cross section, with the waves taken from the 2HD slide for the Campeche 60-km (37.29-mi) slide at the time of maximum inundation (Mid-Friction Case "b"). The general conclusion reached after comparing the East Breaks scenario with the Campeche scenario is that given the level of uncertainty in the source parameters, the approaching wave heights for the hypothetical Campeche scenario are comparable to those of the East Breaks scenario.

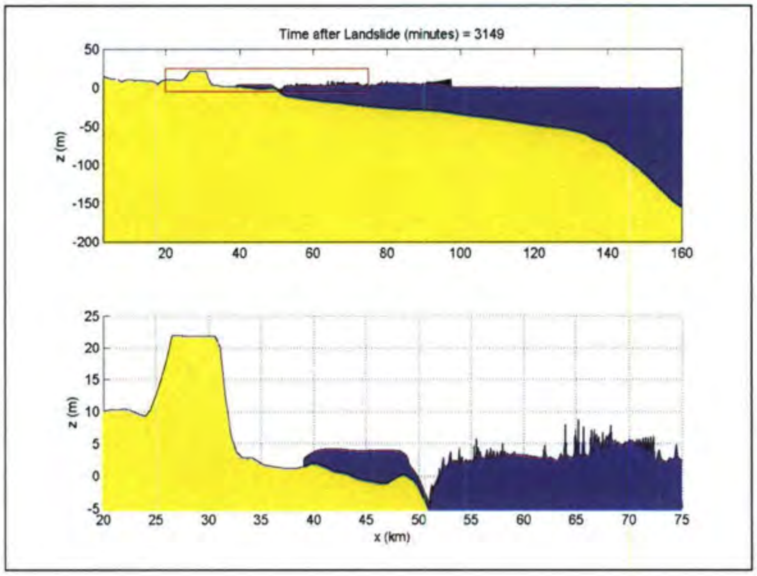

Figure 2.4S.6.4.5-2 Wave Profile at the Time of Maximum Inundation for the Campeche 2HD 60-km Slide Width Source Scenario and for the Mid-Friction Case (Case "b") (Top) View Across the Continental Shelf (Bottom) View Near the STP Site.

An independent analysis of the 10 percent exceedance high tide was conducted for 16 years of NOAA National Ocean Service Center for Operational Oceanographic Product Services (NOS-CO-OPS) data at the Freeport tide gauge station (years 1992 through 2007), (NOAA, 2008). The 10 percent exceedance high tide was determined to be 0.45 m (1.48 ft) relative to the MSL for these years. This finding is consistent with the applicant's estimate of 0.46 m (1.51 ft) relative to the MSL and is indicated in the FSAR, but the number is inconsistent with the estimated 1.08 m (3.54 ft) indicated in the response to RAI 02.04.06-1 (ML083460084). The long-term sea level rise at the Freeport station is 4.35 ± 1.12 mm/year (yr) (0.17 ± 0.04 in./yr), according to the NOAA NOS-CO-OPS data and also indicated in the applicant's RAI response. The estimate in the applicant's FSAR is 5.87 ± 0.74 mm/yr (0.23 ± 0.03 in./yr). Therefore, the PMT water level for the conservative 2HD tsunami during the next century is 4 m (13.12 ft) maximum tsunami runup + 0.45m (1.47 ft) (10 percent exceedance high tide) plus 0.59 m (1.94 ft) (century sea level rise) or approximately a sum of 5.04 m (16.53 ft).

Results of the analysis indicate that the PMT source is a submarine landslide, either along the continental slope directly across from the site (i.e., East Breaks scenario) or along the Campeche Escarpment. There is a high degree of uncertainty in the source parameters for the latter scenario. Hot-start initial conditions were used to represent conservative values related to tsunami generation efficiency. In addition, several bottom-friction parameters for overland flow were tested representing realistic and conservative estimates. Realistic wave propagation in the 2HD simulation, yielded the PMT runup of approximately 5 m (16.44 ft) (relative to the MSL) for conservative hot-start initial conditions and conservative values of bottom friction for overland flow, considering the effects of a 10 percent exceedance high tide and sea level rise during the next century.

2.4S.6.4.5 Effects on Safety-Related Facilities

Information Submitted by Applicant

Because the maximum tsunami water level associated with the PMT is below grade elevations at the site, the applicant concludes that there will be no onsite tsunami waves affecting safety-related facilities.

NRC Staff's Technical Evaluation

NRC staff concurred with the applicant that because the maximum tsunami water level associated with the PMT is below grade elevations at the site, there will be no onsite tsunami waves affecting safety-related facilities.

2.4S.6.5 Post Combined License Activities

There are no post COL activities related to this subsection.

2.4S.6.6 Conclusion

NRC staff reviewed the applicant's submittals in FSAR Section 2.4S.6 and in response to the RAIs. As set forth above, the applicant presents and substantiates sufficient information pertaining to estimates of the effects from probable maximum tsunami hazards at the proposed STP site, and no outstanding information is required to be addressed in the COL FSAR for this section. Furthermore, the applicant considered the most severe natural phenomena that have been historically reported for the site and surrounding area while describing the probable maximum tsunami hazards, with a sufficient margin for the limited accuracy, quantity, and period of time in which the historical data were accumulated.

NRC staff accepted the methodologies used to determine the severity of the tsunami phenomena reflected in this analysis, as documented in this SER section. In the context of the above discussion, the applicant's analysis is acceptable for use in establishing the design bases for SSCs important to safety, as may be proposed in a COL or CP application. Accordingly, the staff concluded that the use of these methodologies results in an analysis containing a sufficient margin for the limited accuracy, quantity, and period of time in which the data were accumulated. Moreover, the PMT surge level estimates by both the applicant and the staff are far below the bounding main cooling reservoir breach water level of 12.2 m (40 ft) MSL or the plant grade of 10.36 (34 ft) MSL, thus the staff concluded that the postulated PMT would not affect the proposed STP site.

Therefore, the staff found that the identification and consideration of the PMT hazards set forth above are acceptable and meet the requirements of 10 CFR 52.79(a)(1)(iii), 10 CFR 100.20(c), and 10 CFR 100.23(d)(3). The information addressing COL Information Item 2.14 is adequate and acceptable.

2.4S.6.7 References

Bourgeois, J., et al., "A Tsunami Deposit at the Cretaceous-Tertiary Boundary in Texas," *Science, 29, Vol 241, pp 567–570*, July 1988.

Chaytor, J.D., et al., "Size distribution of submarine landslides along the U.S. Atlantic Margin," *Marine Geology*, pp. 16–27, 2009.

Geist, E.L., et al., "Technical Letter Report with Open Items for the South Texas Project FSAR Section 2.4.6 Tsunami," submitted to the U.S. Nuclear Regulatory Commission under JCN Q-4151, Task Order No. 2, U.S. Geological Survey Administrative Report (ADAMS Accession No. ML092610497), 2009.

Lynett, P., and P.L.F. Liu, "A numerical study of submarine-landslide-generated waves and runup," *Proceedings of the Royal Society of London, A, Vol. 458: pp 2885–2910*, December 2002.

Lynett, P., et al., "Field survey and numerical simulations: A review of the 1998 Papua New Guinea tsunami," *Pure and Applied Geophysics*, Vol. 160: pp 2119–2146, 2003.

NOAA, "NOS Station #A772440, Freeport – Vertical Historical Tide Data," available at http://tidesandcurrents.noaa.gov/sltrends/sltrends.shtml; accessed on August 19, 2008, ten Brink, U.S., et al., "Evaluation of Tsunami Sources with the Potential to Impact the U.S. Atlantic and Gulf Coasts—An Updated Report to the Nuclear Regulatory Commission," U.S. Geological Survey Administrative Report, 2008.

2.4S.7 Ice Effects

2.4S.7.1 Introduction

This section of the FSAR addresses the ice effects to ensure that safety-related facilities and water supply are not affected by ice-induced hazards.

This SER section presents an evaluation of the following topics based on data provided by the applicant in the FSAR and information available from other sources: ice conditions and historical ice formation, ice jam events, the effect of ice on cooling-water system, and any additional information requirements prescribed in the "Contents of Application" sections of the applicable subparts to 10 CFR Part 52.79(a).

2.4S.7.2 Summary of Application

In Section 2.4S.7, the applicant addresses the information related to the site ice effects. In addition, in this section, the applicant provides site-specific supplemental information to address COL License Information Item 2.16 identified in DCD Tier 2, Revision 4, Section 2.3.

COL License Information Item

- COL License Information Item 2.16 Ice Effects

This section addresses the COL-specific information identified in DCD Tier 2, Revision 4, Section 2.3. COL License Information Item 2.16 requires the COL applicants to demonstrate that safety-related facilities and the water supply are not affected by ice flooding or blockage. This information is provided below.

2.4S.7.3 Regulatory Basis

The associated acceptance criteria for the identification and evaluation of ice effects are in Section 2.4.7 of NUREG-0800.

The applicable regulatory requirements for identifying ice effects are as follows:

- 10 CFR Part 100, as it relates to identifying and evaluating hydrological features of the site. The requirement to consider physical site characteristics in site evaluations is specified in 10 CFR 100.20(c).

- 10 CFR 100.23(d)(3) sets forth the criteria to determine the siting factors for plant design bases with respect to flood level and wave action at the site.

- 10 CFR 52.79(a)(1)(iii), as it relates to identifying hydrologic site characteristics with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

In addition, the staff used the regulatory positions of the following regulatory guides for the identified acceptance criteria:

- RG 1.27, “Ultimate Heat Sink for Nuclear Power Plants”

- RG 1.59, “Design Basis Floods for Nuclear Power Plants,” as supplemented by best current practices

2.4S.7.4 Technical Evaluation

The staff reviewed the information in Section 2.4S.7 of the STP Units 3 and 4 COL FSAR. The staff's review confirmed that the information in the application addresses the relevant information related to the site ice effects. The staff's technical review of this section includes an independent review of the applicant's information in the FSAR and in the responses to the RAIs.

This section of the SER provides the staff's evaluation of the technical information presented in FSAR Section 2.4S.7.

COL License Information Item

- COL License Information Item 2.16 Ice Effects

The staff reviewed the applicant's information in FSAR Section 2.4S.7. The staff independently assessed the potential for formation of ice at the STP site using available data. The staff's evaluation is described below.

2.4S.7.4.1 Ice Conditions and Historical Ice Formation

Information Submitted by Applicant

The applicant uses long-term daily air temperature data from onsite measurements (1990-2006) and from the Bay City station (1942–2006) to assess the potential for ice formation near the STP site. The maximum cumulative degree-day is a measure of severity of site-specific winter weather conditions conducive to ice formation. The applicant states that, in the observed daily air temperature records at the site, there was only one instance in 1983 when the average daily air temperature was below freezing for five consecutive days. The applicant also states that at the Bay City station there are two instances (1973 and 1989) when the average daily air temperature was below freezing for four consecutive days, and three instances (1948, 1951, 1963, 1985) when the average daily air temperature was below freezing for three consecutive days. Based on these data, the applicant concludes that conditions conducive to freezing rarely occur near the site, and these rare occurrences are of a very short duration.

LCRA recorded water temperature data from 1982 through 2006 at three stations: Bay City (Site 12284), Wharton (Site 12286), and Columbus (Site 12290), which are located approximately 22.5, 59.5, and 114.3 km (14, 37, and 71 mi) from the STP site, respectively. The applicant uses the LCRA recoded data to determine the minimum water temperatures in the Colorado River near the STP site. The minimum recorded water temperature in this data set is 5.1 °C (41.2 °F) on February 6, 1985. At the intake within the main cooling reservoir, water temperatures ranged from 10.6 to 33.4 °C (51.1 to 92.1 °F) based on measurements between 1997 and 2005. Based on these data, the applicant concludes that there is no risk of ice formation near the STP site.

NRC Staff's Technical Evaluation

The staff independently analyzed air temperature data downloaded from the National Climatic Data Center (NCDC) web site (NCDC 2008a, b, c) for three NCDC cooperative stations: Bay City Water Works (Coop ID 410569), Matagorda 2 (Coop ID 415659), and Palacios Municipal Airport (Coop ID 416750). The data at the Bay City station span the periods from October 1909 through July 1917 and from July 1942 through December 2008. The data at the Matagorda station span the period from July 1910 through December 2008. The data at the Palacios station span the period from February 1943 through February 2009. The staff analyzed these data to determine several parameters related to low air temperatures at these stations. These parameters are summarized in Table 2.4S.7-1 below.

Table 2.4S.7-1. Statistics of Low Air Temperatures Near the STP Site

Statistics	Bay City	Matagorda	Palacios
Lowest daily mean air temperature	-8.6 °C (16.5 °F) on 12/23/1989	-7.5 °C (18.5 °F) on 12/23/1989	-8.1 °C (17.5 °F) on 12/23/1989
Number of days with daily mean air temperature below freezing	83 of 24,530	59 of 28,820	63 of 23,934
Longest period with daily mean air temperature at or below 32 °F (occurrences)	5 (twice)	5 (once)	5 (once)
Longest period with daily mean air temperature at or below 18 °F (occurrences)	1 (twice)	0 (none)	1 (once)
°C = degrees centigrade; °F = degrees Fahrenheit.			

Based on the above analysis, the staff concluded that the mean air temperature near the STP site occasionally falls below freezing. However, these spells do not last more than 5 consecutive days. Frazil ice forms in turbulent, supercooled water that is not covered by an ice layer but is directly in contact with the atmosphere when the air temperature is below 18 °F (USACE 2002). The daily mean air temperature at or below -7.8 °C (18 °F) was not sustained for more than a day. The staff concluded that ice formation near the STP site is an unlikely event. The staff also concluded that because of the lack of sustained air temperatures below 18 °F, frazil ice formation is unlikely near the STP site.

2.4S.7.4.2 Ice Jam Events

Information Submitted by Applicant

The applicant states that there are no records of ice jams on the Lower Colorado River in the USACE Ice Jam Database. Review of the water temperature data from 1982 through 2006 shows that water temperatures in the Lower Colorado River never approach freezing. Therefore, the applicant notes that the formation of frazil and anchor ice at the RMPF is highly unlikely. The applicant also states that existence of large dams upstream of the site reduces the possibility that any surface ice or ice flows will move downstream to the STP site.

NRC Staff's Technical Evaluation

The staff searched the USACE Ice Jam Database to locate ice jam and ice dam events on the Colorado River (USACE 2008). There is only one ice jam event listed in the database, and that jam is on the Brazos River at Rainbow, Texas. The weather bureau reported that in 1940, the Brazos River was obstructed by rough ice on January 22 through 23, January 25 through 27, and January 25 through 28. However, there are no records of any ice jam or ice dam formation on the Colorado River in the database.

Based on the Ice Jam Database search, the staff determined that the formation of ice jam and ice dam in the Colorado River near the vicinity of the STP site has never been observed. Therefore, the staff concluded that the formation of ice jam and ice dam near the STP site is an unlikely event.

2.4S.7.4.3 Effect of Ice on Cooling-Water Systems

Information Submitted by Applicant

The applicant states that the UHS for each of STP Units 3 and 4 consists of mechanical draft cooling towers and water-storage basin. The storage basin contains a sufficient capacity to supply water for 30 days following a design-basis accident without the storage basin receiving any makeup water.

The applicant states that the UHS and RSW systems remove heat from the closed-loop reactor building cooling-water system during normal, hot standby, normal shutdown, startup, loss of preferred power, and emergency shutdown operating modes. The UHS is also designed to bypass the cooling towers during cold weather operation. Ice formation in the UHS basin is not expected because of the temperate climatic condition near the site and because of the fact that it is always in service during the above operating modes.

NRC Staff's Technical Evaluation

The staff concurred with the applicant that the storage basin of the UHS is the only safety-related system that could be affected by ice formation. The applicant states that the UHS system will be designed to bypass the cooling tower and use the UHS water-storage basin directly during the cold weather operation. Continuous use of the UHS also reduces the possibility of ice formation within the UHS water-storage basin owing to the emitted heat in the cooling water. Therefore, the staff concluded that ice effects on the UHS are not significant.

2.4S.7.5 Post Combined License Activities

There are no post-COL activities related to this subsection.

2.4S.7.6 Conclusion

The staff performed an independent analysis to determine that ice and frazil formation near the STP site is unlikely. The staff also determined that no historical ice jam or ice dam formation in the Colorado River has been observed upstream or downstream of the site. The staff determined that brief freezing spells near the STP site would not affect the safety-related UHS operation.

As set forth above, the applicant presents and substantiates information relative to the ice effects important to the design and siting of the proposed plant. The staff found that the applicant has considered the appropriate site phenomena for establishing the design bases for SSCs important to safety. The staff accepted the methodologies used to determine the potential for ice formation and blockage reflected in these site characteristics, as documented in SERs for previous licensing actions. Accordingly, the staff concluded that the use of these methodologies results in site characteristics with a margin sufficient for the limited accuracy, quantity, and period of time in which the data were accumulated. Therefore, no outstanding information is required to be addressed in the COL FSAR related to this section.

Based on the above review, the staff concluded that the identified site characteristics meet the requirements of 10 CFR 52.79 and 10 CFR 100.20(c), with respect to establishing the design basis for SSCs important to safety. The information addressing COL Information Item 2.16 is adequate and acceptable.

2.4S.7.7 References

National Climatic Data Center, "Bay City Wtr Wks, Bay City, TX," 2008a; http://www4.ncdc.noaa.gov/cgi-win/wwcgi.dll?wwDI~StnSrch~StnID~20024300; accessed April 17, 2008.

National Climatic Data Center, "Matagorda 2, Matagorda, TX," 2008b; http://www4.ncdc.noaa.gov/cgi-win/wwcgi.dll?wwDI~StnSrch~StnID~20024268; accessed April 17, 2008.

National Climatic Data Center, "Palacios Municipal Airport, Palacios, TX," 2008c; http://www4.ncdc.noaa.gov/cgi-win/wwcgi.dll?wwDI~StnSrch~StnID~20024269; accessed April 17, 2008.

South Texas Nuclear Operating Company, "South Texas Project Combined License Application," Revision 0, Part 2, Final Safety Analysis Report, 2007.

U.S. Army Corps of Engineers, "Engineering and Design – Ice Engineering," Engineer Manual 1110-2-1612, Department of the Army, Washington, D.C., 2002.

U.S. Army Corps of Engineers, "Ice Jam Database," Ice Jam Information Clearinghouse, Cold Regions Research and Engineering Laboratory, 2008; http://www.crrel.usace.army.mil/icejams/; accessed April 17, 2008.

2.4S.8 Cooling-Water Canals and Reservoirs

2.4S.8.1 Introduction

This section of the FSAR addresses the cooling-water canals and reservoirs used to transport and impound water supplied to the safety-related SSCs. This SER section presents an evaluation of the design basis for the capacity and operating plan for safety-related cooling-water canals and reservoirs, and any additional information requirements prescribed in the "Contents of Application" sections of the applicable subparts of 10 CFR Part 52.

2.4S.8.2 Summary of Application

In Section 2.4S.8, the applicant provides site-specific information related to the cooling water canals and reservoirs. In addition, in this section, the applicant provides site-specific supplemental information to address COL License Information Item 2.17 identified in DCD Tier 2, Revision 4, Section 2.3.

The applicant addressed the information related to cooling-water canals and reservoirs as follows:

COL License Information Item

- COL License Information Item 2.17 Cooling-Water Channels and Reservoirs

COL License Information Item 2.17 requires the COL applicants to provide the basis for the hydraulic design of channels and reservoirs used to transport and impound plant cooling and to protect safety-related structures.

2.4S.8.3 Regulatory Basis

The associated acceptance criteria are in Section 2.4.8 of NUREG–0800.

The applicable regulatory requirements for cooling-water canals and reservoirs are as follows:

- 10 CFR Part 100, as it relates to identifying and evaluating hydrological features of the site. The requirement to consider physical site characteristics in site evaluations is specified in 10 CFR 100.20(c).

- 10 CFR 100.23(d)(3), as it sets forth the criteria to determine the siting factors for plant design bases with respect to flood levels and wave actions at the site.

- 10 CFR 52.79(a)(1)(iii), as it relates to identifying hydrologic site characteristics with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

In addition, the staff used the regulatory positions of the following regulatory guides for the identified acceptance criteria:

- RG 1.27, "Ultimate Heat Sink for Nuclear Power Plants"

- RG 1.59, "Design Basis Floods for Nuclear Power Plants," as supplemented by best current practices

- RG 1.102, "Flood Protection for Nuclear Power Plants"

2.4S.8.4 Technical Evaluation

The staff reviewed the information in Section 2.4S.8 of the STP Units 3 and 4 COL FSAR. The staff's review confirmed that the information in the application addresses the relevant information related to the cooling-water canals and reservoirs. The staff's technical review of this section includes an independent review of the applicant's information in the FSAR and in the responses to the RAIs.

This section describes the staff's evaluation of the technical information presented by the applicant in FSAR Section 2.4S.8.

COL License Information Item

- COL License Information Item 2.17 Cooling-Water Channels and Reservoirs

The staff reviewed the applicant's supplemental information relating to the cooling-water canals and reservoirs at the STP site and vicinity. The staff's review of the application is summarized below.

2.4S.8.4.1 Cooling-Water Canals

Information Submitted by Applicant

The circulating-water intake structure for STP Units 3 and 4 will be located on the north dike within the main cooling reservoir. The circulating-water discharge structure for STP Units 3 and 4 will be located on the west side of the circulating-water discharge structure for STP Units 1 and 2 on the north embankment of the main cooling reservoir (see FSAR Figure 2.4S.8-1). The main cooling reservoir has two non-safety-related channels, the discharge and intake channels, which were originally designed for four reactor units. Each of the channels has a bottom width of 304.8 m (1,000 ft) and a side slope of 3:1 (horizontal versus vertical or H:V). The bottom elevation of the intake channel varies from 6.2 to 6.7 m (20.5 to 22.0 ft) MSL, and the bottom elevation of the discharge channel is 6.7 m (22.0 ft) MSL. The intake channel will be locally modified to accommodate the approach channel for the new STP Units 3 and 4 intake structure. No modification of the discharge channel will be necessary. The applicant states that, because the intake and discharge channels are submerged, they are not subject to wind-wave activity.

The spillway channel of the main cooling reservoir delivers any discharge from the reservoir over the spillway to the Colorado River. The channel has a length of approximately 1,591 m (5,220 ft), a width of 36.6 m (120 ft), an average depth of 3.7 m (12 ft), a longitudinal slope of approximately 0.2 percent, and a side slope of 5:1 H:V. No changes to this channel will result from the addition of STP Units 3 and 4. The applicant states that the operation of the main cooling reservoir spillway channel is nonsafety-related.

The existing RMPF that provides makeup water from the Colorado River to the main cooling reservoir will be shared among all four STP units. The RMPF is not a safety-related facility. The RMPF will be upgraded to include additional pumps, screens, and rakes to accommodate the additional makeup water demand for STP Units 3 and 4. The addition of STP Units 3 and 4 will not change the original makeup intake design flow rate of about 34 m^3/s (1,200 cfs).

NRC Staff's Technical Evaluation

The staff determined that the applicant has appropriately identified and described all cooling-water channels. Because there are no safety-related canals proposed for STP Units 3 and 4, the staff omitted the evaluation of these canals.

2.4S.8.4.2 Reservoirs

Information Submitted by Applicant

There are two reservoirs on the STP site: the 28.3 km^2 (7,000-ac) main cooling reservoir, which will be shared among all four units and is part of their closed-loop cooling system; and the 186,152 m^2 (46-ac) ECP, which serves as the UHS for STP Units 1 and 2. The ECP will not be affected by the construction of STP Units 3 and 4 and has no function in their operation.

The main cooling reservoir will be part of the closed-loop CWS to dissipate heat produced from all four units during their normal operations. The Colorado River, via the RMPF, will provide makeup water to the main cooling reservoir to replace water losses due to evaporation, seepage, and blowdown. The main cooling reservoir is enclosed by a compacted clay-filled embankment with an exterior slope of 3:1 H:V and an interior slope of 2.5:1 H:V, with a 76.2 cm (30-in.) thick soil cement lining to prevent erosion. The top of the embankment varies in elevation from 20.1 to 20.5 m (65.8 to 67.1 ft) MSL. An interior dike, constructed of compacted clay lies within the main cooling reservoir to prevent the short-circuiting of discharged warm water to the intake. The reservoir side of the main cooling reservoir embankment and both sides of the interior dyke are lined with 76.2-cm (30-in.) -thick soil-cement to protect against erosion from the wave action. The outside of the peripheral embankment is sodded for erosion protection.

Except for June 6, 1985, when the main cooling reservoir water surface elevation was at 8.4 m (27.7 ft) MSL during its initial filling, the minimum water surface elevation in the main cooling reservoir was 10.2 m (33.4 ft) MSL on November 11, 1987; and the maximum water surface elevation in the main cooling reservoir was 14.5 m (47.6 ft) MSL on July 3, 2003. The normal maximum operating water level for STP Units 1 and 2 is 14.3 m (47.0) ft MSL, which is less than the design normal maximum operating level of 14.9 m (49.0 ft) MSL for the reservoir. The applicant states that, when all four units are in operation, the normal maximum operating water surface elevation will be maintained at 14.9 m (49.0 ft) MSL.

New CWS Intake and Discharge Structures

The new CWS intake structure for STP Units 3 and 4 will be approximately 40 m (130 ft) long and 122 m (400 ft) wide. It will be located on the east slope of the interior dike approximately 107 m (350 ft) south of the existing STP Units 1 and 2 CWS intake structure. The new discharge structure for STP Units 3 and 4 will be located on the north embankment of the main cooling reservoir, approximately 305 m (1,000 ft) west of the existing discharge structure. The new structure will be approximately 18.3 m (60 ft) long and 61 m (200 ft) wide. The applicant states that neither structure is safety-related.

Spillway

The main cooling reservoir spillway is located at its southeast corner and is used to release any water exceeding the normal maximum operating storage. The spillway is a gated concrete ogee with the crest at 12.2 m (40.0 ft) MSL. Four 6-ft wide and 2.9 m (9.5-ft) tall gates are located on top of the ogee crest. The spillway is not a safety-related structure.

To check the safety of the embankment from overtopping, the applicant estimates the maximum water surface elevation within the main cooling reservoir during a local PMP event (STPEGS 2006). The applicant routes the 72-hour storm total precipitation input of 141.5 cm (55.7 in.) through the main cooling reservoir accounting for area and storage curves of the reservoir, operating procedures of the main cooling reservoir spillway, and rating curve of the spillway. The applicant sets the initial water surface elevation in the main cooling reservoir to the normal operating water level of 14.9 m (49 ft) MSL. The applicant estimates the maximum water surface elevation in the main cooling reservoir to be 16 m (52.6 ft) MSL, which is significantly lower than the lowest top elevation of the main cooling reservoir embankment of 20 m (65.8 ft).

Embankment Freeboard

The applicant estimates the embankment freeboard using the PMH sustained wind speeds adjusted to overland wind speeds, as described by the NWS (1979) at eight locations within the main cooling reservoir. The applicant estimates wave height, runup, and wind setup elevations using the methods described by the USACE (2008). The applicant states that the waves are not limited by water depth. Under local PMP-induced flooding in the main cooling reservoir, the applicant estimates the stillwater elevation to be 16 m (52.6 ft) MSL. As recommended in ANSI/ANS-2.8–1992 (ANS 1992), the applicant also estimates wind waves induced by a 2-year wind wave and adds them to the stillwater elevation to obtain a final water surface elevation of 17.79 m (58.38) ft MSL, which is significantly below the lowest top elevation of the main cooling reservoir embankment. Therefore, the applicant concludes that there is sufficient freeboard at the main cooling reservoir.

Seiche in Main Cooling Reservoir

The applicant assumes the PMH passing over the reservoir as the forcing mechanism for a seiche in the main cooling reservoir. The applicant estimates significant wave height induced by the PMH winds to be approximately 4 m (13 ft), with a spectral wave period of 4.7 seconds. The applicant estimates the natural frequency of the main cooling reservoir to be approximately 22 minutes. Because the spectral wave period of the PMH-generated wind waves is significantly smaller than the natural frequency of the main cooling reservoir, the applicant concludes that the energy of the PMH-generated waves will dissipate due to frictional losses and the raised water surface will decrease after each oscillation.

NRC Staff's Technical Evaluation

The staff determined that the applicant has appropriately identified and described the main cooling reservoir and its facilities, which are not safety-related structures. The only safety-related water reservoirs proposed for STP Units 3 and 4 are the two engineered, partially buried UHS water-storage tanks (basins) (FSAR Figures 2.5S.4-49A through 2.5S.4-49D). The two UHS water-storage tanks, one for each proposed unit, will be located south of the respective units. Section 9.2.5 of the FSAR evaluates the capacity of these UHS water-storage tanks. There it has been determined that the these UHS water-storage tanks will be sufficient to meet 30 days of the UHS cooling requirement under design-basis accident conditions, without needing a makeup or blowdown. Therefore the staff has determined that the applicant's description of the reservoirs is acceptable.

Embankment Freeboard

During the review of in the main cooling reservoir embankment freeboard, the staff issued RAI 02.04.08-1 requesting the applicant to provide details of estimates of wind setup, wave height, and runup elevations at eight locations along the main cooling reservoir embankment. In a letter dated August 27, 2008 (ML091910403), the applicant's response to RAI 02.04.08-1 states that there will be no physical changes to the main cooling reservoir as a result of the construction and operation of Units 3 and 4 that will affect the characteristics of wind-wave setup and runup. The applicant therefore notes that the original main cooling reservoir freeboard analysis carried out for the design of the main cooling reservoir embankment during the licensing of STP Units 1 and 2 is still valid. In addition, the applicant provides a re-analysis of the wave setup and runup estimates using two conservative scenarios as described below.

The first scenario is the combined event of a 72-hour local PMP over the main cooling reservoir coincident with the 2-year wind wave. By routing the excess water in the reservoir through the spillway, the applicant estimates the maximum reservoir level from the 72-hour PMP of 16 m (52.6 ft) MSL. Using the estimated stillwater level with 2-year wind and an average reservoir bottom elevation of 7 m (23 ft) MSL, the applicant estimates the maximum water level of 17.8 m (58.4 ft) MSL near the spillway and 17.77 m (58.3 ft) MSL at the northern embankment, respectively.

The staff reviewed the applicant's response and determined that the combined event and the method used to estimate the maximum water surface elevation within the main cooling reservoir for the first scenario are appropriate. The STP Units 1 and 2 FSAR, Revision 13 (STPEGS 2006), states that the main cooling reservoir embankment elevation near the spillway and at the north embankment is 20.2 m (66.2 ft) MSL. Therefore, staff determined that the combined event of a 72-hour local PMP event and a 2-year wind wave will not overtop the main cooling reservoir embankment.

The second scenario consists of wind waves induced by PMH winds, with the starting water surface elevation in the main cooling reservoir at the normal operating level of 14.9 m (49 ft) MSL. The applicant states that this analysis was performed for the main cooling reservoir embankment freeboard design during the licensing of STP Units 1 and 2 (STPEGS 2006). STPEGS obtained the PMH speed of from NWS Technical Report 23 (NWS 1979) and adjusted the speed for the movement over land and subsequent open water in the main cooling reservoir. The resulting PMH speed was 66.2 m/s (148 mph).

Subsequently, STPEGS estimated the wind setup from the PMH using the approach described by Saville et al., (1962) and the corresponding wave runup using the approach described by

USACE (1977). STPEGS estimated the maximum water surface elevation along the main cooling reservoir embankment to be 19.9 m (65.2 ft) MSL and noted that it occurs on the south embankment, where the embankment elevation is 20.4 m (66.9 ft) MSL. STPEGS noted that a water surface elevation of 19.3 m (63.4 ft) MSL along the north embankment, where the embankment elevation is 20.2 m (66.2 ft) MSL. Based on this information, the applicant states that the minimum available freeboard along the main cooling reservoir embankment for this scenario is 0.52 m (1.7 ft).

The applicant has modified the FSAR text to reflect the revised analyses in FSAR Subsection 2.4S.8.2.3. The applicant states that FSAR Figures 2.4S.8-2 through 2.4S.8-5 will be deleted. However, in FSAR Revision 4, these figures are not yet deleted. This will be tracked as Confirmatory Item 02.04.08-1.

The staff independently estimated the PMH from NWS Technical Report 23 (NWS 1979), as described in Section 2.4S.5 of this SER. The staff found that the maximum PMH wind speed computed with the SLOSH model near the location of the STP Units 3 and 4 power block is approximately 83.1 m/s (186 mph). The staff independently estimated the wind-wave setup and runup at three locations: the spillway, the south embankment, and the north embankment of the main cooling reservoir. The average depth of the main cooling reservoir is estimated as the difference between normal main cooling reservoir water surface elevation 14.9 m (49 ft) MSL and the average main cooling reservoir bottom elevation 7 m (23 ft) MSL. The staff used a PMH wind speed of 83.1 m/s (186 mph), an initial water surface elevation in the main cooling reservoir of 14.9 m (49 ft) MSL, an average water depth of 7.9 m (26 ft), and 2.5H:1V for the inner slope of the main cooling reservoir embankment. The staff estimated the wind-wave parameters using the USACE (2008) methods. The staff determined that the wind waves within the main cooling reservoir are fetch limited, and the PMH winds are also limited by water depth. USACE (2008) recommends limiting wave heights to 0.6 times the depth of the water body. Therefore, the staff estimated the PMH wind-wave height in the main cooling reservoir to be approximately 4.8 m (15.6 ft) (i.e., 0.6 × 7.9 m [26 ft]). The corresponding estimated wind setups and wave runups using USACE (2008) at the three locations are in Table 2.4S.8-1, below.

Table 2.4S.8-1. NRC Staff-Estimated PMH Wind Setup and Wave Runup at Three Locations Within the Main Cooling Reservoir

Location	**Fetch (km) / (mi)**	**Depth-Limited Wave Height (m) / (ft)**	**Spectral Wave Period (sec)**	**Wind Setup (m) / (ft)**	**Wave Runup (ft)**	**Water surface Elevation (m MSL) / (ft MSL)**
Spillway	5.5 / 3.4	4.8 / 15.6	4.35	0.98 / 3.2	3.41 / 11.2	19.3 / 63.4
North Embankment	5.8 / 3.6	4.8 / 15.6	4.42	1.04 / 3.4	3.47 / 11.4	19.4 / 63.8
South Embankment	5.3 / 3.3	4.8 / 15.6	4.29	0.94 / 3.1	3.35 / 11.0	19.2 / 63.1
MSL = mean sea level						

The staff estimates of water surface elevations within the main cooling reservoir at the three locations are 19.3, 19.4, and 19.2 m (63.4, 63.8, and 63.1 ft) MSL, respectively. The corresponding top elevations of the main cooling reservoir embankment at these locations are

20.2, 20.2, and 20.4 m (66.2, 66.2, and 66.9 ft) MSL, respectively. Therefore, the staff concluded that the PMH wind waves within the main cooling reservoir would not overtop the main cooling reservoir embankment.

The STPEGS Units 1 and 2 UFSAR Subsection 2.4.8.2.3, "Embankment Freeboard," lists the maximum water surface elevation along the south embankment as 65.2 ft MSL, under the effects of PMH winds acting on a normal main cooling reservoir stillwater surface elevation of 14.9 m (49 ft) MSL. STP Units 3 and 4 FSAR Subsection 2.4S.8.2.3, "Embankment Freeboard," states that the maximum water level due to wave runup under PMH winds is an estimated 17.79 m (58.38 ft) MSL. The staff issued RAI 02.04.08-2 requesting the applicant to explain the difference between these two estimates.

In a letter dated August 27, 2008 (ML091910403), the applicant's response to RAI 02.04.08-2 states that the maximum water surface elevation of 19.9 m (65.2 ft) MSL along the south embankment is estimated in the STPEGS Units 1 and 2 FSAR Subsection 2.4.8.2.3 and results from a combination of PMH wind waves on an initial main cooling reservoir stillwater elevation of 14.9 m (49 ft) MSL. The applicant also states that the maximum water surface elevation of 17.79 m (58.38 ft) MSL is estimated at the spillway location based on the combination of a 72-hour local PMP event over the main cooling reservoir (with the initial main cooling reservoir stillwater elevation at 14.9 m [49 ft] MSL) and 2-year winds.

Based on the review of the RAI response 02.04.08-2 and the result of an independent confirmatory analysis, the staff found that the applicant's estimation of the wave setup and runup are adequate. Therefore, the staff concluded that there is sufficient freeboard at the main cooling reservoir and consider RAIs 02.04.08-1 and 02.04.08-2 closed.

<u>Seiche in Main Cooling Reservoir</u>

The staff estimated the spectral wave period of the PMH-induced wind waves within the main cooling reservoir to be approximately 4.4 seconds. The natural period of free oscillation in a rectangular basin of constant depth can be estimated as

$$T = \frac{2L}{\sqrt{gh}}$$

where

T = the period of seiche motion in seconds,

g = the acceleration resulting from gravity (9.8 m/s^2 [32.2 ft/s^2]),

L = the length of the idealized rectangular basin in feet, and

h = the depth of the idealized rectangular basin in feet (Wilson 1972).

The staff used the fetch length to approximate L and the main cooling reservoir average depth to approximate h. The staff estimated the period to vary from 19.9 to 21.7 minutes at the three locations within the main cooling reservoir that were also used to estimate wind-wave setup and runup. Based on the large difference between the natural period of the main cooling reservoir and the spectral wave period of the PMH-induced wind waves, the staff concluded that resonance would not occur within the main cooling reservoir. Therefore, the staff concluded that a wind-induced seiche would not be set up for an extended duration.

Seismic forcing can also generate a seiche within a lake if (1) the period of the seismic wave matches the natural period of free oscillation of the lake, and (2) the seismic waves that have periods not matching the natural period of free oscillation of the lake but provide many cycles of motion over the duration the waves pass the site (Barberopoulou et al., 2006; Barberopoulou 2008). For example, the magnitude 7.9 Denali, Alaska, earthquake of 2002 produced long waves of approximately 100-second periods that produced resonating seiches in lakes near Seattle, Washington (Barberopoulou 2008).

Long or transverse seismic waves that produce horizontal movement can induce seiches within the main cooling reservoir. For example, seiches were set up along the Gulf Coast from seismic surface waves emanating from the M = 9.2 Gulf of Alaska earthquake in 1964, owing in part to the amplification of seismic waves from the thick sedimentary section along the Gulf Coast. It is likely that seismic seiche waves resulting from the 1964 Gulf of Alaska earthquake are nearly the highest possible (refer to this SER Subsection 2.4S.6.4.3), with no significant seismic sources nearby. Therefore, the staff concluded that further review of a seismically induced seiche in the main cooling reservoir is not warranted.

2.4S.8.5 Post Combined License Activities

There are no post-COL activities related to this subsection.

2.4S.8.6 Conclusion

The staff reviewed the application and confirmed that the applicant has addressed the information relevant to design basis for canals and reservoirs used to transport and impound water supplied to the SSCs. In particular, the staff performed an independent confirmatory analysis to determine the potential overtopping of the main cooling reservoir caused by hurricane surge and seiche effects. Based on the result of the confirmatory analysis, the staff concluded that the main cooling reservoir embankment would not be overtopped under PMH or seiche conditions. The staff will track Confirmatory Item 02.04.08-1 to ensure that the proposed changes are included in the next revision of the FSAR.

The staff reviewed the information provided and, for the reasons given above, concluded that the applicant has provided sufficient details about the site description to allow a staff evaluation, as documented in Section 2.4S.8 of this SER. Accordingly, the staff concluded that the use of these methodologies results in site characteristics with a margin sufficient for the limited accuracy, quantity, and period of time in which the data were accumulated.

Based on the above information and review, the staff concluded that the identified site characteristics meet the requirements of 10 CFR 52.79 and 10 CFR Part 100.20(c), with respect to establishing the design basis for SSCs important to safety. The information addressing COL Information Item 2.17 is adequate and acceptable.

2.4S.8.7 References

Barberopoulou, A., "A Seiche Hazard Study for Lake Union, Seattle, Washington," *Bulletin of the Seismological Society of America*, 98:4, 1837–1848, 2008.

Barberopoulou, A., et al., "Long-Period Effects of the Denali Earthquake on Water Bodies in the Puget Lowland: Observations and Modeling," *Bulletin of the Seismological Society of America*, 96:2, 519–535, 2006.

National Oceanic and Atmospheric Administration, "Meteorological Criteria for Standard Project Hurricane and Probable Maximum Hurricane Windfields, Gulf and East Coasts of the United States," NOAA Technical Report NWS 23, U.S. Department of Commerce, Washington, D.C., September 1979.

Saville, T. Jr., E. W. McClendon, A. L. Cochran, "Freeboard Allowances for Waters in Inland Reservoirs," *Journal of Waterways & Harbors Division*, ASCE, May 1962.

South Texas Nuclear Operating Company, "South Texas Project Combined License Application," Revision 0, Part 2, Final Safety Analysis Report, 2007.

South Texas Project Electric Generating Station, "STPEGS Updated Final Safety Analysis Report (UFSAR) for Units 1 & 2, Revision 13," 2006.

U.S. Army Corps of Engineers, *Shore Protection Manual,* Third Edition, Coastal Engineering Research Center, Vicksburg, Mississippi, 1977.

U.S. Army Corps of Engineers, "Coastal Engineering Manual – Part II," Engineering Manual 1110-2-1100, Coastal and Hydraulics Laboratory, Engineer Research and Development Center, Vicksburg, MS, 2008.

Wilson, B. W.,"Seiches," *Advances in Hydroscience*, Vol. 8, Academic Press, New York, NY, 1972.

2.4S.9 Channel Diversions

2.4S.9.1 Introduction

This section of the FSAR addresses channel diversions. It evaluates plant and essential water supplies used to transport and impound water supplies to ensure that they will not be adversely affected by stream or channel diversions. The evaluation includes stream channel diversions away from the site (which may lead to a loss of safety-related water) and stream channel diversions toward the site (which may lead to flooding). In addition, in such an event, it must be ensured that alternate water supplies are available to safety-related equipment.

This SER section presents an evaluation of the following specific areas: (1) historical channel migration phenomena including cutoffs, subsidence, and uplift; (2) regional topographic evidence that suggests a future channel diversion may or may not occur (used in conjunction with evidence of historical diversions); (3) thermal causes of channel diversion, such as ice jams, which may result from downstream ice blockages that may lead to flooding from backwater or upstream ice blockages that can divert the flow of water away from the intake; (4) potential for forces on safety-related facilities or the blockage of water supplies resulting from channel migration-induced flooding (flooding not addressed by hydrometeorologically induced flooding scenarios in other sections); (5) potential of channel diversion from human-induced causes (i.e., land-use changes, diking, channelization, armoring, or failure of structures); (6) alternate water sources and operating procedures; (7) potential effects of seismic and nonseismic information on the postulated worst-case channel diversion scenario for the proposed plant site; (8) any additional information requirement prescribed in the "Contents of Application" sections of the applicable subparts of 10 CFR Part 52.

2.4S.9.2 Summary of Application

In Section 2.4S.9, the applicant describes site-specific information related to the channel diversions. In this section, the applicant provides site-specific supplemental information to address COL License Information Item 2.18 identified in DCD Tier 2, Revision 4, Section 2.3.

<u>*COL License Information Item*</u>

- COL License Information Item 2.18 Channel Division

COL License Information Item 2.18 requires the COL applicants to provide site-specific information related to channel diversion for the STP site. The following information addresses this subject.

2.4S.9.3 Regulatory Basis

The associated acceptance criteria are described in Section 2.4.9 of NUREG–0800.

The applicable regulatory requirements for identifying and evaluating channel diversions are as follows:

- 10 CFR Part 100, as it relates to identifying and evaluating hydrological features of the site. The requirement to consider physical site characteristics in site evaluations is specified in 10 CFR 100.20(c).

- 10 CFR 100.23(d)(3), as it sets forth the criteria to determine the siting factors for plant design bases with respect to flood levels and wave actions at the site.

- 10 CFR 52.79(a)(1)(iii), as it relates to identifying hydrologic site characteristics with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and time in which the historical data have been accumulated.

In addition, the staff used the regulatory positions of the following regulatory guides for the identified acceptance criteria:

- RG 1.27, "Ultimate Heat Sink for Nuclear Power Plants"

- RG 1.59, "Design Basis Floods for Nuclear Power Plants," as supplemented by best current practices

- RG 1.102, "Flood Protection for Nuclear Power Plants"

2.4S.9.4 Technical Evaluation

The staff reviewed the information in Section 2.4S.9 of the STP Units 3 and 4 COL FSAR. The staff's review confirmed that the information in the application addresses the relevant information related to the channel diversions. The staff's technical review of this section includes an independent review of the applicant's information in the FSAR and in the responses to the RAIs. The staff supplemented this information with other publicly available sources of data.

This section describes the staff's evaluation of the technical information presented by the applicant in FSAR Section 2.4S.9.

COL License Information Item

- COL License Information Item 2.18 Channel Division

The staff reviewed the applicant's supplemental information on channel diversions. The staff's review of the applicant's information is summarized below.

2.4S.9.4.1 Historical Channel Diversions

Information Submitted by Applicant

The applicant provides a review of the geology of the STP site vicinity, paleo-geology of the Colorado River Basin, current flow regulation of the Colorado River, and adjacent coastal areas. An examination of the stratigraphic evidence reveals that the Colorado River course near the STP site has maintained its present location for the last 550 years (STPNOC, 2007, FSAR Subsection 2.4S.9.2). The applicant concludes that changes in the present river course due to ice effects and surface faulting are considered unlikely (FSAR Subsections 2.4S.9.3 and 2.4S.9.4.1, respectively). From 1943 to 1973, the land surface in the vicinity of Bay City subsided more than 0.46 m (1.5 ft) because of groundwater withdrawals. However, the applicant finds that groundwater withdrawal rates are declining (FSAR Subsection 2.4S.9.4.2).

Regulation by dams has minimized channel modification during floods (FSAR Subsections 2.4S.9.4.3 and 2.4S.9.5.2). Because Hurricane Carla caused no long-lived channel diversion, channel diversion due to coastal storms is considered unlikely (FSAR Subsection 2.4S.9.4.4).

The applicant states that sand and gravel mining in the Colorado River have taken place near Austin and subsequently the river has eroded new channel paths through abandoned pits in Travis and Colorado counties (FSAR Subsection 2.4S.9.5). However, the applicant states that severe bed degradation in the Lower Colorado River has not been observed. Dredging operations and channel stabilization in the Lower Colorado River have reportedly increased the bank full capacity of the river near the STP Units 3 and 4 site (FSAR Subsections 2.4S.9.5.2 and 2.4S.9.5). The applicant concludes that there is little likelihood of major channel diversions affecting STP Units 3 and 4 safety facilities (FSAR Subsection 2.4S.9.5).

NRC Staff's Technical Evaluation

The applicant's response to RAI 02.04.09-1 dated July 2, 2008 (ML081890239), states that the flood of 1935 had a peak discharge of almost 14,158 m^3/s (500,000 cfs). The applicant also states that the 1935 event was the last major flood to divert a significant flow of the Colorado River into the headwaters of the Tres Palacios Creek. The applicant argues that dams built upstream of the STP site in the Colorado River Basin provide flood control that has greatly reduced major flooding in lower portions of the basin.

The staff reviewed the applicant's response and determined that it is adequate. The applicant's response is consistent with the staff's independent review of historical floods in the Lower Colorado River Basin, as described in Section 2.4S.2 of this SER. The staff used this information when reviewing potential channel diversions of the Colorado River. The staff determined that the applicant's description of historical channel diversions is acceptable. Therefore RAI 02.04.09-1 is closed.

2.4S.9.4.2 Stratigraphic Evidence

Information Submitted by Applicant

The applicant states that stratigraphic evidence in the Colorado River and Caney Creek basins near the STP site suggests that the river has occupied its present course for more than 550 years. The most likely avulsion point on the Colorado River in the future is between Eagle Lake, Texas, and Wharton, Texas (Blum and Valastro Jr., 1994), where the modern Colorado River channel and the abandoned Caney Creek meander belt split within the same valley. Downstream of Wharton, the stream courses of the Colorado River and Caney Creek diverge until they reach the Gulf, separated by approximately 40 km (24.9 mi).

NRC Staff's Technical Evaluation

The staff reviewed the applicant's information in the FSAR and cited references for the stratigraphic data. The staff found no particular evidence of a potential diversion of the Colorado River. Furthermore, the Colorado River is currently highly regulated by upstream dams. Although the lower portions of the river have low relief, flood discharges into the channel near the STP site are greatly reduced since the construction of Mansfield Dam, making the diversion of the Colorado River unlikely.

2.4S.9.4.3 Ice Causes

Information Submitted by Applicant

The applicant considers channel diversion caused by ice jams unlikely on the Colorado River, because there are no historical records of any major rivers in Texas freezing.

NRC Staff's Technical Evaluation

The staff reviewed air temperature data near the STP site in Section 2.4S.8 of this SER and determined that ice formation is an unlikely event near the STP site. The staff also determined that no historical record of ice jam or ice dam formation on the Colorado River exists. The staff therefore concluded that ice is an unlikely cause of channel diversion near the STP site.

2.4S.9.4.4 Flooding of the Site Due to Channel Diversion

Information Submitted by Applicant

There are no reports of channel diversion upstream of the Balcones Escarpment near Austin, Texas. In the vicinity of the STP site the topography is flat, with an average dip of less than 1 degree in regional geologic units. The low slope also indicates a low probability of slope failure. There are no capable faults in the STP site region where surface faulting can occur to induce a slope failure leading to channel diversion.

Ground subsidence of 0.12 ft in the vicinity of Bay City, Texas, was measured between 1918 and 1951 (Hammond, Jr. 1969). Between 1943 and 1973, the land subsidence due to groundwater withdrawals in Matagorda County was 0.46 m (1.5 ft), which is attributed to increased groundwater use after 1940 (Ratzlaff 1982). The Texas Water Development Board (2006) documented a decline in ground water use in Matagorda County, from 47.6 million m^3 (38,554 ac-ft) in 1980 to 46.3 million m^3 (37,537 ac-ft) in 1990 and 17.8 million m^3 (14,413 ac-ft) in 1997. This reduction in the withdrawal of ground water in Matagorda County should also minimize further subsidence.

A large flood or a series of large floods caused by upstream dam failures or significant changes in sea level could result in channel diversion in an unregulated Colorado River Basin. Because regulation in the basin since 1938 has helped to reduce the flood peak discharges, this mechanism of channel diversion is considered unlikely.

In 1961, Hurricane Carla partly obliterated the Matagorda peninsula, but the damage was soon repaired naturally by shoreline sediment migration and deposition (Hyde 2001). The applicant concludes that hurricane effects are not considered a significant cause for channel diversion because even Hurricane Carla, which was a Category 5 hurricane, did not cause any channel diversions in the area (STPNOC 2007).

Downstream of Austin, Texas, sand and gravel mining in the Colorado River have created pits. During flooding, the river has carved new paths through these abandoned pits at several locations in Travis and Colorado counties resulting in artificial cutoffs of historical meanders, and some localized downstream bank effects (Saunders 2002). Although unconstrained gravel mining may lead to severe degradation downstream (Parker 2008), none has been observed in the lower Colorado River. Therefore, the applicant concludes that the gravel mining effect will not contribute significantly to channel diversion near the STP site.

NRC Staff's Technical Evaluation

Because of low reliefs in the lower Colorado River Basin near the STP site, slope failures along the bank of the Colorado River are unlikely. There is also no potential for land subsidence or sand and gravel mining that may divert the course of the Colorado River. The effects of dam failure and hurricanes are evaluated in sections 2.4.4 and 2.45 respectively. Therefore the staff concluded that such events will not divert the Colorado River toward the STP site.

2.4S.9.4.5 Human-Induced Changes of Channel Diversion

Information Submitted by Applicant

A major log jam in the Colorado River had existed for a long time, from its earliest reference in 1690 to its first survey in 1824, where the log jam extended 74 km (46 mi) in length. The jam was finally removed in 1929 during a large flood assisted by earlier, manual removal efforts. The Colorado River delta, which was 182,105 m^2 (45 ac) in 1908, grew to 14 km^2 (3,470 ac) in 1933, 19.8 km^2 (4,890 ac) in 1936, 28.7 km^2 (7,098 ac) in 1941, and 29.1 km^2 (7,200 ac) in 1953. In 1936, a channel cut through the Matagorda peninsula relieved upstream flooding, and caused the river to discharge directly into the Gulf. The creation of upstream dams on the Colorado River has limited sediment delivery to the mouth and as a consequence, the delta has been receding.

During the flood of 1935, flow from the Colorado River was diverted into Tres Palacios Creek and the Tres Palacios Bay. The unregulated river may still be subject to such diversions (Wadsworth, 1966).

The USACE dredged the Colorado River from river kilometer 35 (river mile 22) to the Intracoastal Waterway to stabilize the river planform. The USACE deposited the dredged material along the river on both banks and enclosed by embankments. During this activity, the USACE also filled in the abandoned river channel north of the STP site in the vicinity of Selkirk Island. Because of these measures, the applicant considers a shifting of the river near the STP site unlikely.

NRC Staff's Technical Evaluation

The staff reviewed the applicant's information related to efforts to clear a long-existing log jam in the Colorado River. The USACE also periodically carries out maintenance in the Colorado River to keep the channel navigable. There are no major projects proposed on the Colorado River upstream and downstream of the STP site that may affect its present course. Based on the above review, the staff concluded that human-induced changes in the course of the Colorado River are minor, and the river will therefore not migrate from its present course.

2.4S.9.4.6 Potential of Future Channel Migration and Impact

Information Submitted by Applicant

Because of the presence of control structures upstream of the STP site on the Colorado River and the plan for stabilization measures on the lower Colorado River, the applicant concludes that channel diversion near the STP site is unlikely and will not produce a flood approaching the magnitude of the PMF discussed in FSAR Section 2.4S.2.

NRC Staff's Technical Evaluation

The discharge in the Colorado River near the STP site is highly regulated by upstream dams. There are no major projects proposed for the Colorado River upstream and downstream of the STP site. Therefore, the staff concluded that future channel migration of the Colorado River near the STP site is unlikely.

2.4S.9.5 Post Combined License Activities

There are no post-COL activities related to this subsection.

2.4S.9.6 Conclusion

The staff reviewed the application and confirmed that the applicant has addressed the information to demonstrate that the characteristics of the site fall within the site parameters specified in the DC rule, and no outstanding information is required to be addressed in the COL FSAR related to this section.

As set forth above, the applicant has presented and substantiated information to establish the site description ensuring that the plant and essential water supplies will not be adversely affected. The staff reviewed the information provided and concluded, for the reasons given above, that the applicant has provided sufficient details to address COL License Information Item 2.18. Therefore, the staff concluded that the applicant has met the relevant requirements of 10 CFR 52.79(a)(1) and 10 CFR Part 100 with respect to determining the acceptability of the site.

2.4S.9.7 References

Blum, M. D. and S. Valastro, Jr., "Late Quaternary Sedimentation, Lower Colorado River, Gulf Coastal Plain of Texas," *Geological Society of America Bulletin*, 106:1002–1016, 1994.

Hammond, Jr., W.W., "Ground-Water Resources of Matagorda County, Texas," Texas Water Development Board Report 91, March 1969.

Hyde, H. W., "Soil Survey of Matagorda County, Texas," United States Department of Agriculture, Natural Resources Conservation Service, 2001.

Mace, R.E., et al., eds., "Aquifers of the Gulf Coast of Texas," Texas Water Development Board Report 365, February 2006.

Parker, G., "Transport of Gravel and Sediment Mixtures," Chapter 3, in ASCE Manual 110, *Sedimentation Engineering: Processes, Measurements, Modeling, and Practice*, Garcia, M., ed., American Society of Civil Engineers, p. 1150, 2008.

Ratzlaff, K.W., "Land-Surface Subsidence in the Texas Coastal Region," Texas Water Development Board Report 272, November 1982.

Saunders, G.P., "Impacts of Sand and Gravel Mining on Physical Habitat of the Colorado River and Tributaries, Central Texas," Transactions of the Gulf Coast Association of Geological Societies, pp. 883–890, 2002.

South Texas Nuclear Operating Company, "South Texas Project Combined License Application," Revision 0, Part 2," Final Safety Analysis Report, 2007.

Wadsworth, A.H., "Historical Deltation of the Colorado River, Texas," in *Deltas in Their Geologic Framework*, Shirley, M-L. ed., Houston Geological Society, 1966.

2.4S.10 Flooding-Protection Requirements

2.4S.10.1 Introduction

This section of the FSAR addresses the design bases required to ensure that safety-related facilities will be capable of surviving all design-basis flood conditions, and those of structures and components required for protection of safety-related facilities. This section also describes various types of flood protection used and the emergency procedures to be implemented where applicable.

This SER section provides an evaluation of the following specific areas: (1) safety-related facilities exposed to flooding; (2) type of flood protection (e.g., "hardened facilities," flood doors, bulkheads, etc.) provided for the SSCs exposed to flooding; (3) emergency procedures needed to implement flood-protection activities and warning times available for their implementation reviewed by the organization responsible for reviewing issues related to plant emergency procedures; (4) potential effects of seismic and non-seismic information on the postulated flood-protection for the proposed plant site; and (5) any additional information requirements prescribed in the "Contents of Application" sections of the applicable subparts of 10 CFR Part 52.

2.4S.10.2 Summary of Application

In Section 2.4S.10, the applicant addresses the need for site-specific information on flood-protection requirements. In this section, the applicant provides site-specific supplemental information to address COL License Information Item 2.19 identified in DCD Tier 2 Revision 4, Section 2.3.

Tier 1 Departure

- STP DEP T1 5.0-1 Site Parameters

The applicant identifies the departure of the site-specific maximum flood level from the ABWR DCD standard plant site design maximum flood level parameter, as described in DCD Tier 1, Table 5.

COL License Information Item

- COL License Information Item 2.19 Flooding Protection Requirements

COL License Information Item 2.19 requires the COL applicants to provide site-specific information related to flood protection for the STP site.

2.4S.10.3 Regulatory Basis

The associated acceptance criteria are in Section 2.4.10 of NUREG–0800.

The applicable regulatory requirements for identifying and evaluating flood-protection requirements are as follows:

- 10 CFR Part 100, as it relates to identifying and evaluating hydrological features of the site. The requirement to consider physical site characteristics in site evaluations is specified in 10 CFR 100.20(c).

- 10 CFR 100.23(d)(3), as it sets forth the criteria to determine the siting factors for plant design bases with respect to flood levels and wave action at the site.

- 10 CFR 52.79(a)(1)(iii), as it relates to identifying hydrologic site characteristics with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

In addition, the staff used the regulatory positions of the following regulatory guides for the identified acceptance criteria:

- RG 1.27, "Ultimate Heat Sink for Nuclear Power Plants"

- RG 1.59, "Design Basis Floods for Nuclear Power Plants," as supplemented by best current practices

- RG 1.102, "Flood Protection for Nuclear Power Plants"

- RG 1.206, "Combined License Applications for Nuclear Power Plants (LWR Edition)"

In addition, in accordance with Section VIII, "Processes for Changes and Departures," of "Appendix A to Part 52--Design Certification Rule for the U.S. Advanced Boiling Water Reactor," the applicant identifies one Tier 1 departure requiring prior NRC approval. This departure is subject to the requirements of 10 CFR Part 52, Appendix A, Section VIII.A.4. .

2.4S.10.4 Technical Evaluation

The staff reviewed the information in Section 2.4S.10 of the STP Units 3 and 4 COL FSAR. The staff's review confirmed that the application addresses the relevant information related to the flooding-protection requirements. The staff's technical review of this section includes an independent review of the applicant's information in the FSAR and in the responses to the RAIs. The staff supplemented this information with other publicly available sources of data.

This section describes the staff's evaluation of the technical information presented by the applicant in FSAR Section 2.4S.10.

<u>*Tier 1 Departure*</u>

- STP DEP T1 5.0-1 Site Parameters

The staff's evaluation of this departure as it relates to design-basis flood and protection of safety-related systems is discussed below.

COL License Information Item

- COL License Information Item 2.19 Flooding Protection Requirements

The staff reviewed the applicant's supplemental information on flooding-protection requirements. The staff's review of the application is summarized below.

Information Submitted by Applicant

The applicant states in FSAR Section 2.4S.2 that the design-basis floodwater elevation in the STP Units 3 and 4 power block area is 12.2 m (40 ft) MSL, which is higher than the proposed site grade in the power block area that ranges from 9.8 to 11.2 m (32 to 36.6 ft) MSL. Therefore, the applicant states that all safety-related SSCs of the proposed STP Units 3 and 4 will require flood protection to the design-basis floodwater elevation of 12.2 m (40 ft) MSL, and identifies this in Departure STP DEP T1 5.0-1.

The applicant, under COL Information Item 2.19, identifies safety-related SSCs requiring flood protection. The applicant states that safety-related SSCs for STP Units 3 and 4 include the reactor buildings, control buildings, the UHS water-storage basins, the UHS cooling towers, and RSW pump houses. The applicant adds that these facilities are designed to withstand the combination of flooding conditions and wave runup, including both static and dynamic flooding forces associated with the flooding events, and that the foundations of these facilities are deep enough to withstand the erosive forces of the main cooling reservoir embankment breach.

The safety-related facilities must remain free from flooding and intrusion of water into areas that contain safety-related equipment. The applicant states that all safety-related facilities in the power block area are watertight below 12.2 m (40 ft) MSL. The applicant states that all watertight doors and hatches open outward and are normally closed position under administrative controls. The applicant adds that all ventilation openings are located above 12.2 m (40 ft) MSL, and that the UHS and RSW pump houses are designed to be watertight below 15 m (50 ft) MSL.

The staff issued Supplemental RAI 02.04.05-11 requesting the applicant to provide additional information regarding the PMSS estimation at the STP site and a possible failure of the main cooling reservoir northern embankment due to erosive action of PMSS waters. The following parts of RAI 02.04.05-11 are relevant to this section of the SER: (8) detailed description of various methods used to estimate current velocities during a PMSS event; (9) a detailed description and justification of simplifying assumptions; (10) conservatively selected current velocities and durations for which these currents will affect the main cooling reservoir northern embankment; and (11) justification, including relevant citations, for the ability of the grass-lined outer face of the main cooling reservoir northern embankment to withstand the current velocities without erosion severe enough to cause an embankment breach. The applicant responded in a letter dated November 22, 2010 (ML103330369).

The applicant describes the erosion protection features of the main cooling reservoir northern embankment. The applicant states that the outer face of the main cooling reservoir northern embankment is grass-lined with a slope of 3 horizontal to 1 vertical. The applicant states that the ADCIRC prediction of the PMSS water surface elevation at the STP site, including wave runup, is 8.9 m (29.3 ft) MSL, which is lower than the grade elevation of 10.4 m (34 ft) MSL at the northern face of the main cooling reservoir northern embankment. Therefore, the applicant concludes that failure of the main cooling reservoir northern embankment from the sloshing and erosive action of PMSS waters is not a credible event. The applicant also describes a more conservative scenario where the PMSS was estimated using the SLOSH model. The applicant states that in this very conservative scenario, SLOSH predicted a stillwater storm surge water surface elevation of 11.7 m (38.5 ft) MSL and the coincident wind-wave action would raise the storm surge water surface elevation to 12.7 m (41.8 ft) MSL. The applicant states that the time history of this very conservative scenario showed that the PMSS water surface elevation would be at 10.4 m (34 ft) MSL (i.e., at site grade) for 80 minutes; at or above 11 m (36 ft) MSL for 50 minutes; and at or above 11.6 m (38 ft) MSL for 25 minutes. The applicant states that significant erosion of the grass-lined north face of the main cooling reservoir northern embankment would not occur during this short amount of time, because a grass surface works well for short-term exposure as plant roots keep soil particles bound together to create a flexible system that deforms without tearing. The applicant also states that the flood-protection levee for Texas City survived a sustained surge and wave attack during Hurricane Ike for many hours without a breach (USACE, 2009). The applicant notes that the main cooling reservoir embankment is similar to but much larger than typical hurricane surge-protection levees that have mostly withstood major hurricanes in the past.

In response to RAI 02.04.05-11 part (8), the applicant states that water will flow past the main cooling reservoir northern embankment under the very conservative PMSS scenario predicted by the SLOSH. The applicant notes that the SLOSH does not output current velocities, but they can be estimated using (1) the area around the STP Units 3 and 4 that experiences the PMSS and matching the volume of water that fills and drains through this area during the PMSS event; (2) using Manning's n and a friction slope estimated by change in water surface elevations; and (3) tracking the PMSS wave-front past the site. The applicant uses all three methods to estimate current velocities during the PMSS event.

In response to RAI 02.04.05-11 part (9), the applicant states that a storm surge that would exceed the STP Units 3 and 4 site grade elevation of 10.4 m (34 ft) MSL is not a credible event. The applicant notes that ADCIRC predictions resulted in a PMSS water surface elevation of 8.9 m (29.3 ft) MSL, which is significantly less than the STP Units 3 and 4 site grade elevation. The applicant also states that even the very conservative predictions from the SLOSH resulted in a PMSS water surface elevation that would inundate only a small portion of the main cooling reservoir northern embankment for a short duration. The applicant concludes that any erosion at the base of the main cooling reservoir northern embankment would not threaten a failure.

In response to RAI 02.04.05-11 part (10), the applicant states that the maximum current velocities estimated using the three methods listed above are 3.5 m/s (11.6 fps), 0.9 m/s (3.1 fps), and 1.9 to 4 m/s (6.2 to 13.2 fps), respectively. The applicant also states that the PMSS flow past the main cooling reservoir northern embankment would occur for a maximum duration of 80 minutes.

In response to RAI 02.04.05-11 part (11), the applicant states that the USACE recommends a design velocity of 1.5 to 2.4 m/s (5 to 8 fps) for stable grass-lined flood channels. The applicant states that the grass-lined main cooling reservoir embankment can be expected to sustain a short exposure to current velocities slightly higher than those assumed in the design of flood channels that have a design life of several decades and would likely be subject to flow durations considerably longer than 80 minutes. The applicant concludes that erosion of the main cooling reservoir northern embankment is unlikely.

NRC Staff's Technical Evaluation

Subsection C.I.2.4.10 of RG 1.206 specifies that "the applicant should describe the static and dynamic consequences of all types of flooding on each pertinent safety-related facility." Additionally, Subsection C.I.2.4.14 of RG 1.206 states that "[i]f the applicant will use emergency procedures . . . appropriate water levels and lead times available should be provided." Subsection C.I.2.4.14 also states that "the applicant should develop specific details on . . . the amount of time available to initiate and complete emergency procedures." To meet the above requirements, the staff issued RAI 02.04.10-1 requesting the applicant to provide severe flood levels in addition to other flood parameters, such as flow velocity and duration (beginning, peak, and end) of inundation important for the design of safety-related SSCs and the preparation of emergency procedures.

As part of the review of COL License Information Item 2.19, the staff asked the applicant to discuss the potential effects on the safety-related facilities of the composition of the flood wave (essentially a mudflow), with respect to the sediment (generated from the gradual breach of the main cooling reservoir embankment) carried with the flow, including dynamic and impact forces. The staff asked the applicant to discuss the conservatism of this case compared to the case presented in the FSAR. The staff postulated that a failure of the main cooling reservoir embankment breach could result in an accumulation of a large amount of bank material at the plant site. The staff asked the applicant to discuss the effects of the settlement of these bank materials around the safety-related structures; the necessary shutdown or operation procedures of the plant after the postulated main cooling reservoir northern embankment failure; and how these effects, if significant, will be addressed in FSAR Section 2.4.14, "Technical Specifications and Emergency Operations Requirements."

The applicant's response to RAI 02.04.10-1 dated November 13, 2008 (ML083250480), states that the entrance-level slab elevation of STP Units 3 and 4 safety-related SSCs is 10.7 m (35 ft) MSL. The applicant also states that the STP Units 3 and 4 site will experience a floodwater surface elevation exceeding 10.7 m (35 ft) MSL under two scenarios: (1) the flood in the power block area under the effects of a local PMP event, and (2) the flood resulting from a postulated breach of the main cooling reservoir northern embankment. Based on Subsection C.I.2.4.14 of RG 1.206, the staff finds this response reasonable acceptable and considers RAI 02.04.10-1 closed.

The applicant reports that using HEC-RAS software to estimate the local PMP flows, the average estimates of cross-sectional velocities within the power block area were between 0.03 to 0.21 m/s (0.1 and 0.7 fps) in the West Channel, which will be located west of STP Unit 4. The applicant states that the average cross-sectional velocities in the East Channel, which will be located east of the STP Unit 3 power block, were between 0.06 to 0.37 m/s (0.2 and 1.2 fps). The applicant reports that the estimated total duration of runoff during the local PMP event was approximately 7 hours in both the West and the East Channels. The applicant also states that the duration of discharges exceeding 28.3 m^3/s (1,000 cfs) was less (1 hour in both the West and the East Channels). The applicant notes that the local PMP event is a slow-moving event that allows the plant operators sufficient time to take action.

In the letter dated January 28, 2009 (ML091880126), the applicant reports that the flood resulting from the main cooling reservoir northern embankment breach was simulated using RMA2, which is a two-dimensional, depth-averaged, hydrodynamic model developed by the USACE (2005). Section 2.4S.4 of this SER describes the staff's review of the flood, erosion, and sedimentation and sediment transport following the postulated breach of the main cooling reservoir northern embankment. The applicant has proposed a site characteristic of 12.2 m (40 ft) MSL for the highest floodwater surface elevation at the STP Units 3 and 4 site.

The applicant also states that the sediment-laden floodwaters will produce a greater force on SSCs compared to non-sediment-laden waters. The applicant reports that the maximum simulated flow velocity is approximately 1.4 m/s (4.7 fps), and the maximum simulated sediment concentration of the flow is 23 kg/m^3. The applicant estimates that the maximum drag force on the projected submerged area of the SSCs would be 214.8 kg/m^2 (44 lb/ft^2).

In Section 2.4S.4 of this SER, the staff postulated a combination of events that could be triggered from erosion of the toe of the main cooling reservoir northern embankment during the PMSS event. Because the applicant had not yet provided an analysis to show whether this is a plausible event, the staff did not confirmed the design-basis flood elevation of 12 m (40 ft) MSL, which is reported in FSAR Section 2.4S.4 and the drag forces on SSCs reported above. This issue was tracked as **Open Item 2.4.10-1** in the SER with open items.

The applicant responded to RAI 02.04.05-11 parts (8) through (11), as described above. The staff reviewed the applicant's submittal. As described in Section 2.4S.5.4 of this SER, the staff determined that the applicant has performed a reasonable and conservative site-specific estimate of the PMSS. The staff agreed with the applicant's conclusion that the maximum PMSS water surface elevation accounting for a wind setup effect would not exceed 8.9 m (29.3 ft) MSL. As described in Section 2.4S.5.4 above, the staff concluded that the maximum PMSS water surface elevation at the STP Units 3 and 4 site accounting for a wind setup and runup would not exceed 9.1 m (30 ft) MSL, 1.2 to 1.8 m (4 to 6 ft) below the STP Units 3 and 4 site grade of 10.4 to 11 m (34 to 36 ft) MSL. The applicant states in FSAR Subsections 2.4S.4.2.2.2.2 and 2.4S.4.2.2.2.4.1 that the terrain immediately downstream of a service road running along the toe of the exterior slope of the main cooling reservoir northern embankment acts as a control against the development of a breach. The applicant states that the terrain elevation at this location is 8.8 m (29 ft) MSL. Because the maximum PMSS water surface elevation including the wind setup and runup effects is 9.1 m (30 ft) MSL, the staff concluded that the lower reach of the toe of the main cooling reservoir northern embankment would experience currents during the PMSS event. The slope of the main cooling reservoir northern embankment at this location is 6 horizontal to 1 vertical (Figure 10 in the applicant's response to RAI 02.04.05-11, ML103330369). Because of the gentle slope and relatively small area of the toe of the main cooling reservoir northern embankment that would be inundated during the PMSS event, the staff concluded that it is unlikely that PMSS currents would cause significant damage to the toe of the northern embankment. For the main cooling reservoir embankment to fail, the erosive action of the PMSS current would have to erode the toe to such an extent that (1) a pipe would form extending to the interior face of the embankment; or (2) an extensive sliding surface would form extending from the downstream to near the upstream face of the embankment. Because the toe of the main cooling reservoir northern embankment is inundated only with a depth of 0.3 m (1 ft) near the exterior end of the embankment, the staff concluded that such a failure mechanism is unlikely. During the PMSS event, the STP Units 3 and 4 power block with a grade elevation of 10.4 to 11 m (34 to 36 ft) MSL would remain unaffected, because the PMSS water surface elevation would not exceed 9.1 m (30 ft) MSL. Therefore, the staff concluded that even in the unlikely scenario that the main cooling reservoir northern embankment were to fail because of erosive action of PMSS currents, the resulting flood would be similar to and not more severe than that analyzed in Section 2.4S.4 of the FSAR and reviewed by the staff in Section 2.4S.4 of this SER. Therefore, the staff determined that the characteristics of the design-basis flood related to Departure STP DEP T1 5.0-1 and the corresponding drag forces on safety-related SSCs are as described in FSAR Section 2.4S.4 and reviewed by the staff in Section 2.4S.4 of this SER.

Because of the reasons stated above, the staff determined that further characterization of a PMSS-induced main cooling reservoir northern embankment failure is not warranted. Therefore, **Open Item 2.4.10-1** is closed.

2.4S.10.5 Post Combined License Activities

There are no post-COL activities related to this subsection.

2.4S.10.6 Conclusion

NRC staff reviewed the application and confirmed that the applicant has addressed the information demonstrating that the characteristics of the site fall within the site parameters specified in the Design Certification (DC) rule, and no outstanding information is required to be addressed in the COL FSAR related to this section.

As set forth above, the applicant has presented and substantiated information relative to the flood protection measures important to the design and siting of this plant. The staff found that the applicant has considered the appropriate site phenomena in establishing the flood protection measures for SSCs. The staff reviewed the applicant's information and, for the reasons stated above, concluded that the applicant, as documented in Section 2.4S.10 of this SER, has provided sufficient details about the site description to allow the staff to evaluate whether the applicant has met the relevant requirements of 10 CFR 52.79(a)(1) and 10 CFR Part 100, with respect to determining the acceptability of the site. The information addressing COL License Information Item 2.19 is adequate and acceptable. The characteristics of the design-basis flood related to Departure STP DEP T1 5.0-1 are described in Section 2.4S.4 of this report.

2.4S.10.7 References

GAO, "Hurricane Protection: Statutory and Regulatory Framework for Levee Maintenance and Emergency Response for the Lake Pontchartrain Project," GAO-06-322T, Washington, DC, 2006.

U.S. Army Corps of Engineers, "User's Guide to SED2D WES," Version 4.5. Coastal and Hydraulics Laboratory, Waterways Experiment Station, Engineer Research and Development Center, Vicksburg, MS, 2003.

U.S. Army Corps of Engineers, "User's Guide to RMA2 WES," Version 4.5, Coastal and Hydraulics Laboratory, Waterways Experiment Station, Engineer Research and Development Center, Vicksburg, MS, 2005.

2.4S.11 Low Water Considerations

2.4S.11.1 Introduction

This section of the FSAR addresses natural events that may reduce or limit the available safety-related cooling-water supply. The applicant ensures that an adequate water supply will exist to shut down the plant under conditions requiring safety-related cooling.

This SER section provides an evaluation of the following specific areas: (1) low-water conditions due to the worst drought considered reasonably possible in the region; (2) the effects of low water surface elevations caused by various hydrometeorological events and a potential blockage of intakes by sediment, debris, littoral drift, and ice because they can affect the safety-related water supply; (3) the effects of low water on the intake structure and pump design bases in relation to the events described in SAR Sections 2.4.7, 2.4.8, 2.4.9, and 2.4.11, which consider the range of water supply required by the plant (including minimum operating and shutdown flows during anticipated operational occurrences and emergency conditions) compared with availability (considering the capability of the UHS to provide adequate cooling water under conditions requiring safety-related cooling); (4) the use limitations imposed or under discussion by Federal, State, or local agencies authorizing the use of the water; (5) the potential effects of seismic and non-seismic information on the postulated worst-case low-water scenario for the proposed plant site; and (6) any additional information requirements prescribed in the "Contents of Application" sections of the applicable subparts of 10 CFR Part 52.

2.4S.11.2 Summary of Application

In Section 2.4S.11, the applicant addresses the impacts of low water on safety-related water supply. In this section, the applicant provides site-specific supplemental information to address COL License Information Item 2.20, identified in DCD Tier 2, Revision 4, Section 2.3.

The applicant addresses the information as follows:

<u>*COL License Information Item*</u>

- COL License Information Item 2.20 Cooling-Water Supply

COL License Information Item 2.20 requires the COL applicants to provide site-specific information related to the cooling-water supply for the STP site. The following information addresses this subject.

2.4S.11.3 Regulatory Basis

The acceptance criteria, are in Section 2.4.11 of NUREG–0800.

The applicable regulatory requirements for identifying the effects of low water are as follows:

- 10 CFR Part 100, as it relates to identifying and evaluating hydrological features of the site. The requirement to consider physical site characteristics in site evaluations is specified in 10 CFR 100.20(c).

- 10 CFR 100.23(d)(3), as it sets forth the criteria to determine the siting factors for plant design bases with respect to seismically induced floods and water waves at the site.

- 10 CFR 52.79(a)(1)(iii), as it relates to identifying hydrologic site characteristics with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

In addition, the staff used the regulatory positions of the following regulatory guides for the identified acceptance criteria:

- RG 1.27, "Ultimate Heat Sink for Nuclear Power Plants"

- RG 1.59, "Design Basis Floods for Nuclear Power Plants," as supplemented by best current practices

2.4S.11.4 Technical Evaluation

The staff reviewed the information in Section 2.4S.11 of the STP Units 3 and 4 COL FSAR. The staff's review confirmed that the information in the application addresses the relevant information related to the low-water considerations. The staff's technical review of this section includes an independent review of the applicant's information in the FSAR and in the responses to the RAIs. The staff supplemented this information with other publicly available sources of data.

This section describes the staff's evaluation of the technical information presented by the applicant in FSAR Section 2.4S.11.

COL License Information Item

- COL License Information Item 2.20 Cooling-Water Supply

The staff issued RAI 02.04.11-1 requesting the applicant to provide details to support the following statement in FSAR Subsection 2.4S.11.6 or to delete the statement if it is not relevant here: "The potential effects of all site-related proximity, seismic, and non-seismic information on the postulated worst-case low-flow scenario for the proposed plant site have been considered in establishing the design basis."

In a letter dated June 26, 2008 (ML081970231), the applicant's response to RAI 02.04.11-1 states that the statement is not relevant to FSAR Section 2.4S.11. The applicant has removed the statement from the FSAR. The staff is satisfied with this change and therefore RAI 02.04.11-1 is considered closed.

2.4S.11.4.1 Low Flow in Rivers and Streams

Information Submitted by Applicant

The STP Units 3 and 4 site is located on the west bank of the Colorado River at river kilometer 23.5 (river mile 14.6). Tidal influence reaches upstream to river kilometer 35.4 (river mile 22). An inflatable dam 1.6 km (1 mi) downstream from Bay City and immediately upstream from the USGS gauge station at Bay City (08162500) is used to maintain water quality for irrigation withdrawals. Discharge data at this station are available from 1948 but are affected by the presence of upstream dams.

Zero daily discharge was recorded 13 times from 1951 to 1956. During June and July of 1967, irrigation withdrawals reduced the downstream flow to less than 0.028 m^3/s (1 cfs) for 58 days. The recorded minimum 1-day and 7-day low flows are 0 and 0.014 m^3/s (0 and 0.5 cfs), respectively.

The primary source of makeup water to the UHS water-storage basins will be onsite ground water wells that are unaffected by low-flow conditions in the Colorado River. The main cooling reservoir will provide a backup source of UHS makeup water for the UHS.

<u>NRC Staff's Technical Evaluation</u>

The staff reviewed the applicant's information in the FSAR. There is a separate UHS for each STP Units 3 and 4 that is configured with a dedicated, partially buried water-storage basin sized to hold sufficient water to provide cooling following a design-basis accident and to maintain safe shutdown conditions for 30 days, without requiring any makeup or blowdown. Also, the staff confirmed that the Colorado River water will not be used as a source of UHS makeup. Therefore, the staff determined that low flow in river and streams will not affect the safe operation of STP Units 3 and 4.

2.4S.11.4.2 Low Water Resulting from Surges, Seiches, or Tsunamis

<u>Information Submitted by Applicant</u>

The applicant proposes ground water wells as the primary source of makeup water to the UHS water-storage basin. Ground water conditions are not expected to be affected by low water resulting from surges, seiches, or tsunamis. Formation of ice jams or ice dams near the STP site is unlikely based on historical air and water temperature observations near the STP site. Therefore, the applicant concludes that low water resulting from ice-induced causes is unlikely.

<u>NRC Staff's Technical Evaluation</u>

The staff reviewed the applicant's information in the FSAR. There is a separate UHS for each STP Units 3 and 4 that is configured with a dedicated, partially buried water-storage basin sized to hold sufficient water to provide cooling following a design-basis accident and to maintain safe-shutdown conditions for 30 days, without requiring any makeup or blowdown. Therefore, the staff determined that low water resulting from surges, seiches, or tsunamis will not affect the safety of STP Units 3 and 4.

2.4S.11.4.3 Historical Low Water

<u>Information Submitted by Applicant</u>

The most severe drought event on record, based on observations from 1898 through 2004, is the 10-year drought that spanned from May 1947 to April 1957.

The inflatable dam below Bay City, which was installed in 1963, regulates low flow in the Colorado River. During extremely low-flow conditions in the Colorado River, the river water surface elevation near the RMPF is expected to be approximately equal to the tidal elevation to prevent saltwater intrusion.

NRC Staff's Technical Evaluation

The staff reviewed the applicant's information in the FSAR. The staff's review determined that the applicant has provided sufficient information and the description of the historical low water is adequate and acceptable.

2.4S.11.4.4 Future Controls

Information Submitted by Applicant

The safety-related systems of STP Units 3 and 4, including the UHS, do not depend on the Colorado River or the main cooling reservoir as water sources directly. Ground water is the primary source of makeup water to the UHS basins. The units will be shut down when the water surface elevation in the main cooling reservoir drops below 7.8 m (25.5 ft) MSL. At this elevation, the main cooling reservoir contains 47.1 million m^3 (38,150 ac-ft) of water, which exceeds the 30-day UHS makeup water requirements.

NRC Staff's Technical Evaluation

The staff reviewed the applicant's information in the FSAR. There is a separate UHS for each of STP Units 3 and 4 that is configured with a dedicated, partially buried water-storage basin sized to hold sufficient water to provide cooling following a design-basis accident and to maintain safe shutdown conditions for 30 days, without requiring any makeup or blowdown.

Based on this information, the staff determined that the development of any future controls on the Colorado River water or on ground water supplies will not have an adverse effect on the safety-related water held in the dedicated UHS water-storage basins for STP Units 3 and 4.

2.4S.11.4.5 Plant Requirements

Information Submitted by Applicant

The RSW and the UHS systems provide essential cooling during normal operation, normal shutdown, emergency shutdown, testing, and loss of preferred power while maintaining the temperature of the UHS water basin at or below 35 °C. The water-storage basins for the UHS (one each for STP Units 3 and 4) are designed with sufficient capacity to provide cooling during shutdown and cooldown and to maintain safe-shutdown conditions for 30 days, without the need for any makeup or blowdown. Water from the UHS basins is lost because of natural and forced evaporation, drift, seepage, and blowdown. The primary sources of makeup water to the UHS basins are site wells. The main cooling reservoir is the secondary source of makeup water provided to the basins through the turbine service-water system.

NRC Staff's Technical Evaluation

The staff reviewed the applicant's information in the FSAR. There is a separate UHS for each of STP Units 3 and 4 that is configured with a dedicated, partially buried water-storage basin sized to hold sufficient water to provide cooling following a design-basis accident and to maintain safe shutdown conditions for 30 days, without requiring any makeup or blowdown.

Based on this information, the staff determined from the applicant's information in the FSAR that the primary sources of makeup water to the UHS water-storage basins are site wells. The main cooling reservoir, via the turbine service-water system, will be used as the secondary source of makeup water to the UHS water-storage tanks.

2.4S.11.4.6 Heat Sink Dependability Requirements

Information Submitted by Applicant

The UHS water-storage basins are sized to hold sufficient water to provide cooling and to maintain a safe shutdown following a design-basis accident for 30 days, without any reliance on makeup water.

NRC Staff's Technical Evaluation

The staff reviewed the applicant's information in the FSAR. There is a separate UHS for each of STP Units 3 and 4 that is configured with a dedicated, partially buried water-storage basin sized to hold sufficient water to provide cooling following a design-basis accident and to maintain safe shutdown conditions for 30 days, without requiring any makeup or blowdown.

2.4S.11.5 Post Combined License Activities

There are no post-COL activities related to this subsection.

2.4S.11.6 Conclusion

The staff reviewed the applicant's information in the FSAR and supplemented that with observations from the staff's site audit and other publicly available data sources. The STP Units 3 and Unit 4 will each have an engineered, partially buried water-storage tank. These UHS water-storage tanks will be designed to hold sufficient water to provide cooling following a design-basis accident and to maintain a safe shutdown for a period of 30 days, without makeup or blowdown. The makeup water for the two UHS storage basins will come from site wells, which are the primary source, and from the main cooling reservoir, which is the secondary source. The staff determined that low-water events in the vicinity of the STP Units 3 and 4 site will not affect their safe operation. Therefore, no outstanding information is required to be addressed in the COL FSAR related to this section.

As set forth above, the applicant presents and substantiates information relative to the low-water effects important to the design and siting of this plant. The staff reviewed the available information and concluded, for the reasons given above, that the identification and consideration of the potential for low-water conditions are acceptable and meet the requirements of 10 CFR 52.79, 10 CFR 100.23(d)(3), and 10 CFR 100.20(c), with respect to determining the acceptability of the site.

Therefore, the staff found that the applicant has considered the appropriate site phenomena in establishing the design bases for SSCs important to safety. The staff accepted the methodologies used to determine the potential for low-water conditions, as reflected in these design bases and documented in SERs for previous licensing actions. Accordingly, the staff concluded that the use of these methodologies results in design bases containing a margin sufficient for the limited accuracy, quantity, and period of time in which the data were accumulated. The staff concluded that the identified design bases meet the requirements of 10 CFR 52.79, 10 CFR 100.23(d)(3), and 10 CFR 100.20(c), with respect to establishing the design basis for SSCs important to safety. The information addressing COL Information Item 2.20 is adequate and acceptable.

2.4S.11.7 References

South Texas Nuclear Operating Company, "South Texas Project Combined License Application," Revision 0, Part 2, Final Safety Analysis Report, 2007.

2.4S.12 Groundwater

2.4S.12.1 Introduction

This section of the FSAR describes the hydrogeological characteristics of the site. The most significant objective of ground water investigations and monitoring at this site is to evaluate the effects of ground water on safety-related plant facilities. The evaluation is performed to ensure that the maximum ground water elevation remains below the DCD site parameter value. The other significant objectives are to examine whether the ground water provides any safety-related water supply, determine whether dewatering systems are required to maintain ground water elevation below the required level, measure characteristics and properties of the site needed to develop a conceptual site model of ground water movement, and estimate the direction and velocity of movement of potential radioactive contaminants.

This SER section provides a review of the following specific areas: (1) description and onsite ground water use, (2) ground water source, (3) subsurface pathways, (4) monitoring and safeguard requirements, and (5) site characteristics for subsurface hydrostatic loading.

2.4S.12.2 Summary of Application

In Section 2.4S.12, the applicant addresses ground water conditions in terms of influences on structures and water supply. In addition, the applicant provides site-specific supplemental information to address COL License Information Item 2.32 identified in DCD Tier 2, Revision 4, Section 2.3.

COL License Information Item

- COL License Information Item 2.32 Effect of Groundwater

This COL license information item directs the applicant to provide site-specific information that addresses ground water conditions in terms of influences on structures and water supply. Specifically, the DCD states that COL applicants (1) "will analyze the groundwater condition for the specific site," and (2) "will evaluate the effect of groundwater on such site geotechnical properties as total and effective unit weights, cohesion and angle of internal friction, and dynamic soil properties." This section of the FSAR addresses the first of these subtopics, and FSAR Section 2.5.4 addresses the second subtopic.

2.4S.12.3 Regulatory Basis

The relevant requirements of the Commission regulations for ground water and the associated acceptance criteria are described in Section 2.4.12 of NUREG–0800.

The applicable regulatory requirements are as follows:

- 10 CFR Part 100, as it relates to identifying and evaluating hydrological features of the site. The requirement to consider physical site characteristics in site evaluations is specified in 10 CFR 100.20(c).

- 10 CFR 100.23(d)(3), as it sets forth the criteria to determine the siting factors for plant design bases with respect to seismically induced floods and water waves at the site.

- 10 CFR 52.79(a)(1)(iii), as it relates to identifying hydrologic site characteristics with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

2.4S.12.4 Technical Evaluation

The staff reviewed the information in Section 2.4S.12 of the STP Units 3 and 4 COL FSAR. The staff's review confirmed that the information in the application addresses the relevant information related to the ground water. The staff's technical review of this section includes an independent review of the applicant's information in the FSAR and in the responses to the RAIs. The staff supplemented this information with other publicly available sources of data.

This section describes the staff's evaluation of the technical information presented by the applicant in FSAR Section 2.4S.12.

COL License Information Items

- COL License Information Item 2.32 Effect of Groundwater

The staff reviewed the applicant's supplemental response on ground water. The staff's review of the application is summarized below.

The staff's discussion of ground water characteristics is organized into the following technical areas. Unresolved RAIs and open items are described where appropriate within these areas.

2.4S.12.4.1 Regional Hydrogeologic Description

Information Submitted by Applicant

In FSAR Subsection 2.4S.12.1, the applicant describes the geologic formations, regional and local ground water aquifers, aquifer recharge and discharge regions, and onsite ground water use. The applicant formulates a hydrogeologic conceptual model of the STP site using four different data sources that include the following:

- a desktop study of the regional ground water system derived from State, Federal, and other sources of information

- a review of STP Units 1 and 2 documentation with regard to ground water

- the collection of site-specific geotechnical, geologic, and hydrogeologic data for STP Units 3 and 4

- the evaluation of site-specific hydrogeology through regional data and information

The applicant considers site-specific STP Units 3 and 4 data in the context of site-specific STP Units 1 and 2 data and regional data to formulate the conceptual model for the STP site, with a focus on the proposed location for STP Units 3 and 4.

In FSAR Subsection 2.4S.12.1.1, the applicant describes the STP site as being in Matagorda County, Texas, and within the Gulf Coastal Plains physiographic province of the Coastal Prairies sub-province. The applicant describes the Coastal Prairies sub-province as follows. The Coastal Prairies sub-province is a broad band paralleling the Texas Gulf Coast (Ryder 1996). The sub-province is characterized by relatively flat topography ranging from sea level at the coast to 91 m (300 ft) MSL along the northern and western inland boundaries of the sub-province. Underlying the STP site is a wedge of southeasterly dipping sedimentary deposits ranging in age from Holocene (i.e., 10,000 years ago to present) through Oligocene (i.e., 33.9 million to 23 million years before present).

In FSAR Subsection 2.4S.12.1.2, the applicant describes the Coastal Lowlands Aquifer System underlying the STP site. Within Texas, the term Gulf Coast Aquifer is used to describe this aquifer system (Mace et al., 2006). Numerous local aquifers are found in the thick sequence of alternating and interfingering beds of clay, silt, sand, and gravel. Ground water ranging in quality from fresh to saline is found in these sediments. Three depositional environments are evident: continental (alluvial plain); transitional (delta, lagoon, beach); and marine (continental shelf). Oscillations of the ancient shorelines have resulted in overlapping mixtures of sediments. The Texas nomenclature shown by the applicant in FSAR Figure 2.4S.12-5 is used to describe the aquifer system underlying the site. The common regional hydrogeologic unit names are as follows:

- Chicot Aquifer
- Evangeline Aquifer
- Burkeville Confining Unit
- Jasper Aquifer
- Catahoula Confining Unit
- Vicksburg-Jackson Confining Unit.

The applicant describes the Gulf Coast Aquifer (referred to here as the regional aquifer) as extending to either its contact with the top of the Vicksburg-Jackson Confining Unit or the depth where groundwater contains a total dissolved solids (TDS) concentration greater than 10,000 mg/L (0.78 lb/ft^3) [Ryder 1996]). The regional aquifer system is recharged by precipitation falling on the aquifer outcrop areas along the northern and western boundaries of the physiographic province. It discharges through evapotranspiration, the loss of water as the base flow into streams, discharge into the Gulf of Mexico, and water pumped from groundwater wells.

In FSAR Subsection 2.4S.12.1.3, the applicant describes the hydrogeology of the Chicot Aquifer underlying Matagorda County. In this vicinity, the aquifer extends from the land surface to a depth of more than 304.8 m (1,000 ft). The stratigraphic units that compose the Chicot Aquifer in this vicinity, from the land surface downward, include the Holocene alluvium of the river valley; the Pleistocene age (i.e., from 1.8 M years ago to approximately 10,000 years ago) Beaumont, Montgomery, and Bentley Formations; and the Willis Sand. In general, the groundwater flows toward the south and southeast and the Gulf of Mexico. However, river channel incisions can act as localized areas of recharge or discharge and result in varied groundwater flow directions.

NRC Staff's Technical Evaluation

The staff reviewed FSAR Subsections 2.4S.12.1.1, 2.4S.12.1.2, and 2.4S.12.1.3. The staff's review confirmed that the applicant has addressed relevant information. In its review of the application, the staff found the applicant's information comparable to that in documents on the hydrology and aquifers of the region by the USGS (Ryder, 1996; Ryder and Ardis 2002); the TWDB (Hammond, 1969; Mace et al., 2006); and the LCRA (Young et al., 2007). Based on the above information, the staff concluded that the applicant's descriptions of the regional hydrogeologic setting, regional ground water aquifers, and the local hydrogeology are accurate.

2.4S.12.4.2 Site-Specific Hydrogeology

Information Submitted by Applicant

In FSAR Subsection 2.4S.12.1.4 and the applicant's proposed revision in the responses to RAI 02.04.12-28 dated November 23, 2009 (ML093310392), the applicant describes the Chicot Aquifer underlying the STP site as an aquifer divided into two aquifer units: the Shallow Aquifer and the Deep Aquifer. The Shallow Aquifer is recharged a few miles north of the STP site and discharges into the alluvial material of the Colorado River east of the site and into groundwater wells. The Deep Aquifer is recharged farther north in Wharton County at aquifer outcrops and discharges to groundwater wells, the Colorado River estuary, and Matagorda Bay. In general, the ground water quality of the Deep Aquifer is superior to that of the Shallow Aquifer and consequently, the Deep Aquifer is the primary ground water production zone.

The applicant notes that the Shallow Aquifer can be subdivided into an Upper Shallow Aquifer and a Lower Shallow Aquifer. The applicant states that it completed 28 ground water observation wells in the Upper and Lower Shallow Aquifer during initial site characterization activities and completed an additional 26 observation wells in 13 well clusters during July and August of 2008 (ML092710096), as described in RAI 02.04.12-28 dated November 23, 2009 (ML093310392). The initial 28 observation wells were designed and located to supplement the existing STP network and provide a basis for estimating hydraulic gradients and determine the plausible current and future groundwater flow directions. Among the wells, several are designed to provide vertical hydraulic gradient data. The additional 26 wells supplement the aquifer data and better resolve alternative pathways originating in the vicinity of the main cooling reservoir and the proposed power block. The applicant also collected piezometric data monthly from December 2006 through 2007 and quarterly throughout 2008. Data collected since September 2008 include all 54 wells.

The applicant states that site characterization data collected for the proposed STP Units 3 and 4 confirmed and expanded the understanding of the aquifers that underlie the STP site. FSAR Table 2.4S12-14 and its proposed changes in RAI responses (ML101390277 and ML093310392) is reproduced here as Tables 2.4S.12-1 and 2.4S.12-2 in this SER to show the representative thickness of the hydrogeologic units and the properties of confining layers and aquifers in the STP hydrogeologic profile. In sequence from the ground surface to the deepest aquifer affected directly by the plant operation are the following units and thicknesses:

- Upper Shallow Aquifer confining layer, 6.1 m (20 ft)
- Upper Shallow Aquifer, 7.6 m (25 ft)
- Lower Shallow Aquifer confining layer, 6.1 m (20 ft)
- Lower Shallow Aquifer, 12.2 m (40 ft)

- Deep Aquifer confining layer, 30.5 m (100 ft)
- Deep Aquifer, 152.4 m (500 ft)

Currently, there are five completed STP production wells; the deepest reach 213 m (700 ft) below ground surface (BGS). There is some communication between the Upper and Lower Shallow Aquifers, but there appears to be little communication between the Shallow and Deep Aquifers.

The applicant acknowledges that the main cooling reservoir influences the hydraulic head within the Upper Shallow Aquifer; however, the applicant has concluded based on recently collected piezometric data presented in the response to RAIs dated September 21, 2009 (ML092710096) and November 23, 2009 (ML093310392), that there is no obvious mounding in the Lower Shallow Aquifer from the main cooling reservoir. Potentiometric surface maps of the Upper and Lower Shallow Aquifers are presented (1) in the COL FSAR, (2) in the applicant's groundwater model report dated November 30, 2009 (ML093360350), and (3) in the applicant's supplemental response to RAI 02.04.12-28 dated November 23, 2009 (ML093310392). Maps of the Shallow Aquifer potentiometric surfaces are also provided in the UFSAR for STP Units 1 and 2 (STPEGS 2006).

Table 2.4S.12-1. Representative Properties of Confining Layers in the STP Hydrogeologic Strata (from FSAR Table 2.4S.12-14 and its proposed revision in the RAI response dated November 23, 2009).

Hydrogeologic Unit	Property	Units	Representative Value*	Range	FSAR Source
Upper shallow aquifer confining layer	Thickness	m (ft)	6.1 (20) (pj)	3.1-9.1 (10–30)	Figure 2.4S.12-20
	Vertical hyd cond	m/s (gpd/ft^2)	1.9E-09 (0.004) (gm)	2.4E-10-2.4E-08 (0.0005–0.05)	Table 2.4S.12-13
	Bulk (dry) density	kg/m^3 (pcf)	1,618 (101)	1,544-1,841 (96.4–114.9)	Table 2.4S.12-12
	Total porosity	%	40	31.8–42.8	Table 2.4S.12-12
Lower shallow aquifer confining layer	Thickness	m (ft)	6.1 (20) (pj)	4.6-7.6 (15–25)	Figure 2.4S.12-20
	Vertical hyd gradient	-	0.29	0.02–0.29	Table 2.4S.12-8
	Vertical hyd cond	m/s (gpd/ft^2)	1.9E-09 (0.004) (gm)	2.4E-10-2.4E-08 (0.0005-0.05)	Table 2.4S.12-13
	Bulk (dry) density	kg/m^3 (pcf)	1,586 (99)	1,398-1,725 (87.3–107.7)	Table 2.4S.12-12
	Total porosity	%	42	36.1–47.2	Table 2.4S.12-12
Deep aquifer confining layer	Thickness	m (ft)	30.5 (100) (pj)	30.5-45.7 (100 –150)	Subsection 2.4S.12.3.1
	Vertical hyd cond	m/s (gpd/ft^2)	1.9E-09 (0.004) (gm)	2.4E-10-2.4E-08 (0.0005-0.05)	Table 2.4S.12-13
Deep aquifer confining layer (cont'd.)	Bulk (dry) density	kg/m^3 (pcf)	1,618 (101)	1,315-1,784 (82.1–111.4)	Table 2.4S.12-12
	Total porosity	%	41	33.4–51.8	Table 2.4S.12-12

*Values are arithmetic mean except where noted.
gm = geometric mean; am = arithmetic mean; pj = professional judgment; hyd = hydraulic; hyd cond =

Hydrogeologic Unit	Property	Units	Representative Value*	Range	FSAR Source
hydraulic conductivity; pcf = pounds per cubic foot.					

Table 2.4S.12-2 Representative Hydrogeologic Properties of Aquifers in the STP Hydrogeologic Strata (from FSAR Table 2.4S.12-14 and its proposed revision in the RAI response dated November 23, 2009).

Hydrogeologic Unit	Property	Units	Representative Value*	Range	FSAR Source
Upper Shallow Aquifer; Piezometric Surface 5 to 10 ft BGS	Thickness	m (ft)	7.6 (25) (pj)	6.1-9.1 (20–30)	Figure 2.4S.12-20
	Transmissivity	m^2/s (gpd/ft)	5.7E-03 (3,708) (gm)	1.7E-03-1.9E-02 (1,100–12,500)	Table 2.4S.12-10
	Storage coefficient	-	1.2E-03	1.7E-03 – 7E-04	Table 2.4S.12-10
	Horizontal hyd cond	m/s (gpd/ft^2)	7.8E-05 (165) (gm)	3.1E-05-2.0E-4 (65–420)	Table 2.4S.12-10
	Horizontal hyd gradient	-	0.002 (southeast) 0.0008 (southwest)	0.0007–0.002; 0.0005–0.0008	Subsection 2.4S.12.2.2
	Bulk (dry) density	kg/m^3 (pcf)	1,586 (99)	1,557-1,605 (97.2–100.2)	Table 2.4S.12-12
	Total porosity	%	41	39.5–41.7	Table 2.4S.12-12
	Effective porosity	%	33	31.6–33.4	Table 2.4S.12-12
Lower Shallow Aquifer; Piezometric Surface 10 to 15 ft BGS	Thickness	m (ft)	12.2 (40) (pj)	7.6-15.2 (25–50)	Figure 2.4S.12-20
	Transmissivity	m^2/s (gpd/ft)	2.8E-02 (18,209) (gm)	2.0E-02-5.1E-02 (13,000–33,150)	Table 2.4S.12-10
	Storage coefficient	-	5.8E-4	4.5E-4–7.1E-4	Table 2.4S.12-10
Lower Shallow Aquifer; Piezometric Surface 10 to 15 ft BGS (cont'd)	Horizontal hyd cond	m/s (gpd/ft^2)	2.6E-04 (543) (gm)	1.9E-04-3.1E-04 (410-651)	Table 2.4S.12-10
	Hydraulic gradient	-	0.0007 (southeast)	0.0004–0.0007	Subsection 2.4S.12.2.2;
	Bulk (dry) density	kg/m^3 (pcf)	1,634 (102)	1,514-1,922 (94.5–120.0)	Table 2.4S.12-12
	Total porosity	%	39	28.8–43.9	Table 2.4S.12-12
	Effective porosity	%	31	23.0–35.1	Table 2.4S.12-12
Deep Aquifer	Thickness	m (ft)	243.8-304.8 (800-1000) (pj)		Ryder (1996), LCRA (2007b)
	Transmissivity	m^2/s (gpd/ft)	4.9E-02 (31,379) (gm)	3.7E-02-7.7E-02 (24,201–50,000)	Table 2.4S.12-10
	Storage coefficient	-	4.9E-4	2.2E-4–7.6E-4	Table 2.4S.12-10
	Horizontal hyd cond	m/s (gpd/ft^2)	2.0E-04 (420) (gm)	4.9E-05-1.9E-03 (103–3,950)	Table 2.4S.12-9
	Hydraulic gradient	-	0.002	0.0008–0.002	Subsection 2.4S.12.2.2
	Bulk (dry) density	kg/m^3 (pcf)	1,634 (102)	1,514-1,922 (94.5–120.0)	Lower Shallow Aquifer
	Total porosity	%	39	28.8–43.9	Lower Shallow Aquifer
	Effective porosity	%	31	23.0–35.1	Lower Shallow Aquifer

Hydrogeologic Unit	Property	Units	Representative Value*	Range	FSAR Source
*Value = arithmetic mean except where noted. gm = geometric mean; am = arithmetic mean; pj = professional judgment; hyd = hydraulic; hyd cond = hydraulic conductivity; pcf = pounds per cubic foot.					

NRC Staff's Technical Evaluation

The staff reviewed FSAR Subsection 2.4S.12.1.4 and the proposed revision in the response to RAI 02.04.12-28 dated November 23, 2009 (ML093310392). The staff confirmed that the applicant has addressed the relevant information. The staff's review of the application included documents on the hydrology and aquifers of the site; the Units 1 and 2 UFSAR (STPEGS 2006); the staff's final environmental statements related to the operation of STP Units 1 and 2 (NRC 1975, 1986); and the applicant's responses to the RAIs cited above.

The applicant completed documenting additional characteristics of the site as a result of RAIs and commitments made during the acceptance review of the application. The staff's review found that the main cooling reservoir does influence the Upper Shallow Aquifer. The staff also noted that the backfilled excavation at STP Units 1 and 2 does influence the Upper and Lower Shallow Aquifers, and the pathways from proposed STP Units 3 and 4 will need to account for a similar influence at the backfilled excavation of the proposed units. A review of pre-site and site-startup conditions in the Lower Shallow Aquifer, as exhibited in the Units 1 and 2 UFSAR Revision 13, Figures 2.4.13-17 and 2.4.13-17a (STPEGS, 2006) compared to current piezometric levels and contours (see FSAR Figure 2.4S.12-19) and the applicant's proposed changes in the RAI response dated November 23, 2009 (ML093310392), led the staff to raise the issue that there are site influences on the Lower Shallow Aquifer. This issue is addressed in detail in Section 2.4S.12.4.7 of this SER.

The staff reviewed the FSAR and its proposed revisions in response to RAI 02.04.12-28 dated November 23, 2009 (ML093310392). The staff found the applicant's description of site-specific hydrogeology acceptable for the following reasons: (1) the description of the proposed site for STP Units 3 and 4 is consistent with the description of the hydrology and aquifers underlying existing STP Units 1 and 2, and (2) the site characterization provides additional information on the aquifers underlying the proposed site of STP Units 3 and 4. The FSAR changes in response to RAI 02.04.12-28 were tracked as **Confirmatory Item 02.04.12-1** in the SER with open items. The staff verified that the applicant has incorporated the proposed changes in the FSAR. Therefore, RAI 02.04.12-28 and Confirmatory Item 02.04.12-1 are closed.

2.4S.12.4.3 Groundwater Sources and Sinks

Information Submitted by Applicant

In FSAR Subsection 2.4S.12.1.5, the applicant describes the recharge and discharge areas of the regional ground water system. Natural regional ground water flow in the Beaumont Formation (i.e., including the Shallow Aquifer and Deep Aquifer) is from recharge areas northwest of the site toward the Gulf of Mexico or Colorado River alluvium. The main cooling reservoir also recharges the Upper Shallow Aquifer, as demonstrated by potentiometric levels that decrease in piezometers farther from the embankment. A series of 770 relief wells that penetrate the Upper Shallow Aquifer at the toe of the embankment was installed to capture at least 50 percent of the seepage from the main cooling reservoir, as stated in RAI 02.04.12-20 dated December 30, 2008 (ML083660390). Based on site characterization data, the applicant believes that the main cooling reservoir affects the ground water flow direction in the Upper Shallow Aquifer, but the applicant does not detect any obvious mounding in the Lower Shallow Aquifer from the main cooling reservoir as stated in RAI 02.04.12-28 dated September 21, 2009.

The applicant describes the main cooling reservoir recharge to the Upper Shallow Aquifer as occurring mainly as seepage through the reservoir bottom. Design features of the main cooling reservoir dike include a compacted low-permeability clay core, sand drainage blankets, and a series of 770 relief wells completed in the Upper Shallow Aquifer. Ground water flow through the dike and in the underlying aquifer is intercepted, in part, by the system of relief wells. The dikes and the system of relief wells are designed (1) to ensure the stability of the embankment, and (2) to maintain potentiometric levels in the STP Units 1 and 2 power block below the ground surface. During the design of the main cooling reservoir, estimates of total seepage losses and intercepted ground water were 7.031 M m^3/yr (5,700 ac-ft/yr) and 4.75 M m^3/yr (3,850 ac-ft/yr) (i.e., 68 percent intercepted), respectively.

Concentrated pumping from aquifers can alter or locally reverse the regional flow pattern. In the vicinity of the proposed facility, the production wells for existing plants have caused the Deep Aquifer to exhibit a local reversal of the flow pattern. This results in a radial flow toward the production wells from the surrounding aquifer.

In the vicinity of the site, the Holocene age alluvium is recharged by precipitation and by discharge from the Shallow Aquifer. Flow paths in the alluvium are generally short, because flow is intercepted by streams and rivers that incise the alluvial material.

<u>*NRC Staff's Technical Evaluation*</u>

The staff reviewed FSAR Subsection 2.4S.12.1.5 and confirmed that the applicant has addressed relevant information. In the review of the application, the staff also reviewed documents about the hydrology and aquifers of the region from the USGS (Ryder, 1996; Ryder and Ardis 2002); the TWDB (Hammond, 1969; Mace et al., 2006); and the LCRA (Young et al., 2007); in addition to documents submitted by the applicant about the site hydrology and environment (STPEGS, 2006; Reynolds, 2007; Sherwood and Travis 2007, 2008) and documents from the NRC about the site-specific hydrology (NRC, 1975, 1986). The staff concluded that the applicant's description of ground water sources and sinks is consistent with this body of work.

The staff noted the USGS (Ryder, 1996) observations that Matagorda County is in a region of several counties where the greatest amount of groundwater pumping is relatively near the outcrop where the aquifer is recharged and therefore, recharging provides a source to balance the large groundwater withdrawals. This balance was of special interest because of the irrigation of rice in the vicinity of both the pumping and the recharging. Ryder (1996) noted that in areas of little or no pumping, essentially in areas where pre-development conditions persist, the recharge rate is generally between 0 and 2.54 cm/yr (0 and 1 in./yr). During periods of drought, Young et al., (2007) described the average recharge rate as 3.56 to 4.32 cm/yr (1.4 to 1.7 in./yr) and during a wet year, the recharge rate is 11.68 cm/yr (4.6 in./yr). Ryder (1996) also stated that recharge rates increase between 10.2 and 15.2 cm/yr (4 and 6 in./yr) in the rice irrigation areas. As a result, Ryder (1996) concluded that the drawdown was not large in the region (less than 15.2 m [50 ft]) because withdrawals by pumping were balanced by an increase in recharge rates over pre-development levels.

The staff found the applicant's description of the groundwater sources and sinks acceptable because the sources and sinks identified for the Upper and Lower Shallow Aquifer and the Deep Aquifer are consistent with those identified by the USGS, the TWDB, the LCRA, and site-specific documents.

2.4S.12.4.4 Plant Groundwater Use

Information Submitted by Applicant

In FSAR Subsection 2.4S.12.1.6, and its proposed revisions in the response to RAI 02.04.12-28 dated September 21, 2009 (ML092710096), the applicant describes the operation of STP Units 1 and 2 as currently using ground water from five production wells. Annual groundwater usage at STP Units 1 and 2 from 2001 through 2006 was 1.59 M m^3/yr (1,288 ac-ft/yr) (ML092710096). Groundwater use for STP Units 1 and 2 includes supplies for the makeup of the demineralized water system, the potable and sanitary water system, and the fire protection system. Groundwater use for the proposed STP Units 3 and 4 includes similar plant operation water supplies and makeup water to the UHS. The applicant projects (ML092710096) that the normal groundwater consumption rate for the proposed units is 1.94 M m^3/yr (1,575 ac-ft/yr), and the maximum short-term groundwater demand is expected to be as great as 6.83 M m^3/yr (3,434 gpm or 5,547 ac-ft/yr). The groundwater supply wells associated with the proposed STP Units 3 and 4 will not be a safety-related water source because the UHS has a 30-day supply of water, which is sufficient for a plant shutdown without an additional water supply. After studying the plant water use and the site groundwater use issues, the applicant found that the current groundwater use permit limit of 11.1 M m^3 (9,000 ac-ft) during the approximate 3-year permit period is adequate for the operation of STP Units 1 and 2 and the construction, testing, startup, and operation of STP Units 3 and 4.

In FSAR Subsection 2.4S.12.3.3 and its proposed revisions (ML092710096), the applicant describes the proposed groundwater use in light of the existing groundwater permit and groundwater use by the existing STP Units 1 and 2. During the construction of the proposed plant, groundwater will be used for the potable and sanitary water supply, the manufacture and placement of concrete, dust control, backfill moisture, and testing and flushing. During plant operation, groundwater will be used for the potable and sanitary supply, the production of demineralized water, fire protection, and makeup water for the UHS. The groundwater use permit held by the applicant is for 11.1 M m^3 (9,000 ac-ft) during the period of the permit, which is approximately 3 years. For discussion purposes, this use amounts to approximately 3.7 M m^3/yr (3,000 ac-ft/yr) or a normalized continuous pumping rate of 7,040 liters per minute (Lpm) (860 gpm). The relevant sections of the ER in the COL application Part 3, describe details of onsite plant groundwater use and the effects. However, these groundwater uses, including makeup water for the UHS, are not safety-related functions.

NRC Staff's Technical Evaluation

The staff reviewed FSAR Subsection 2.4S.12.1.6 and its proposed revisions. In June 2009, the staff's review of FSAR confirmed that the applicant has addressed the relevant information topics. However, the information in the STP FSAR is not consistent with that found in related sections of the STP ER. RAIs related to the ER (RAI 05.10-04) and the FSAR (RAI 02.04.12-36) were issued to resolve the inconsistencies.

The applicant's responses to ER RAI 05.10-04 dated September 28, 2009 (ML092730285), and to FSAR RAI 02.04.12-36 dated September 21, 2009 (ML092710096), provide ground water use rates for STP Units 1 and 2 under normal and outage conditions, and for STP Units 3 and 4 under normal and maximum conditions. Furthermore, the applicant stated that the existing ground water permit limit provides an adequate water supply for the operation of STP Units 1 and 2 and the construction, initial testing, and operation of STP Units 3 and 4. The applicant stated that the water-storage capacity will be provided to supply the ground water for peak site water demands, and the main cooling reservoir and the Colorado River remain as alternative sources to meet unanticipated peak site water demands.

After reviewing the applicant's responses above and the calculation package on future STP ground water use, the staff concluded that the applicant's description of plant ground water use is accurate. The staff noted that STP ground water wells are not a safety-related source of water for STP Units 3 and 4.

The staff reviewed FSAR Subsection 2.4S.12.3.3 and its proposed revisions dated September 21, 2009 (ML092710096). Based on the applicant's analysis of the groundwater requirements during the construction and operation of the proposed plant, it is apparent that the operation of STP Units 1 and 2 and the construction, testing, and operation of STP Units 3 and 4 can be accomplished using the applicant's currently held groundwater use permit. If additional water is needed to meet maximum short-term groundwater demands for the operation of STP Units 1 and 2 and the construction, testing, and operation of the proposed STP Units 3 and 4, then the main cooling reservoir and Colorado River water are available under the applicant's existing contracts. The applicant states that one or more new groundwater production wells will be constructed to decrease pumping rates at wells, distribute drawdown affects, and ensure a sufficient withdrawal capacity to serve the total site groundwater demand under the existing groundwater permit (ML092710096 and ML092730285). Although specific locations of the new wells have not been provided, the applicant has provided the required separation distances from the existing and proposed reactors and from offsite wells. Ground water supplies for the proposed STP Units 3 and 4 are not safety related.

The staff concluded that the applicant's description of plant ground water use and effects is a consistent and acceptable representation of its intended groundwater use. The FSAR Subsection 2.4S.12.3.3 changes in response to RAI 02.04.12-36 (ML092710096) were tracked as **Confirmatory Item 02.04.12-1** in the SER with open items. The staff verified that the applicant has incorporated the proposed changes in the FSAR. Therefore, RAI 02.04.12-36 and Confirmatory Item 02.04.12-1 are closed.

Historical and Projected Groundwater Use

Information Submitted by Applicant

In FSAR Subsection 2.4S.12.2.1, the applicant describes the historical and projected groundwater uses in Matagorda County. In Section 2.3 of the COL application ER, the applicant also provides details of historical and projected groundwater uses. Table 2.4.12-3 summarizes the quantity of groundwater permitted and the various estimates of the groundwater resource in Matagorda County. The annual quantity of groundwater permitted by the Coastal Plains Groundwater Conservation District (CPGCD) exceeds the current estimates of managed available groundwater and the estimated groundwater supply. The permitted use also exceeds recorded usage within the county. The CPGCD notes that little science has been applied to estimating the managed available ground water resource adopted in the site's ground water management plan (Turner, Collie, and Braden, Inc., 2004), and caution should be exercised in using this value (i.e., 115,528 Lpm [30,520 gpm or 49,221 ac-ft/yr]). It is apparent that satisfying the current annual permitted amount within the CPGCD would require investment in infrastructure, including the construction of wells and delivery systems. Satisfying the future demand level in 2060 would also require investment and could be based on water-conservation strategies and desalination of either sea water or brackish ground water.

The infrastructure is in place at the STP site to fully use its permit limit, and although it has not been fully used to date, it is included in the estimated ground water supply value of 83,992 Lpm ([22,189 gpm or 35,785 ac-ft/yr]). The full STP permit limit is included in the annual permitted value.

Table 2.4S.12-3. Ground Water Resource Estimates for Matagorda County.

Resource Description	L/s (gpm)	m^3/yr (ac-ft/yr)	Reference*
Managed available ground water	1,925 (30,520)	6.1E+07 (49,221)	TC&B 2004, Table 1
Estimated ground water supply	1,400 (22,189)	4.4E+07 (35,785)	TC&B 2004, Table 4
Average ground water use 1980–2000	1,183 (18,746)	3.7E+07 (30,233)	TC&B 2004, Table 2
High ground water use–1988	1,707 (27,055)	5.4E+07 (43,634)	TC&B 2004, Table 2
Low ground water use–1998	554 (8,783)	1.8E+07 (14,165)	TC&B 2004, Table 2
Future demand–Total in 2060	2,556 (40,509)	8.1E+07 (65,331)	LCRWPG 2006
Annual permitted (2008–2010)	2,006 (31,800)	6.3E+07 (51,285)	CPGCD 2009
*TC&B = Turner, Collie, and Braden, Inc.; LCRWPG = Lower Colorado River Water Planning Group; CPGCD = Coastal Plains Groundwater Conservation District			

NRC Staff's Technical Evaluation

The staff reviewed FSAR Subsection 2.4S.12.2.1 and the applicant's response to RAI 02.04.12-36 dated September 21, 2009 (ML092710096). The staff's review confirmed that the applicant has addressed the relevant information. In the review of the application, the staff also reviewed documents on Texas water law (Texas Water Code Chapter 36, Groundwater Conservation Districts) and water management documents at the local, regional, and State levels (Turner, Collie, and Braden, Inc., 2004; LCRWPG, 2006, 2009; TWDB, 2006).

After the site audit in 2008, the applicant was asked to revisit the topic of historical and projected groundwater uses. Information in the application was thought to be dated and as such, it might not have reflected the current groundwater use and availability in Matagorda County. The applicant's response to RAI 02.04.12-04 dated July 2, 2008 (ML081890239), which requested groundwater use projections for the region that are consistent with the license period, was reviewed and accepted by the staff. The values cited in this response for available groundwater, 60.71 M m^3/yr (49,221 ac-ft/yr); average groundwater use between 1980 and

2000, 37.29 M m^3/yr (30,233 ac-ft/yr); and available groundwater supply, 44.14 M m^3/yr (35,785 ac-ft/yr) are from the agency responsible for assessing the groundwater resources in Matagorda County and for issuing groundwater use permits (i.e., the Coastal Plains Groundwater Conservation District [Turner, Collie, and Braden, Inc. 2004]). At this time, these values are current and as issued by the authorized body.

The staff concluded that the applicant's description of historical and projected ground water use is an accurate representation of ground water use in the vicinity of the STP site.

2.4S.12.4.5 Groundwater Flow Directions

Information Submitted by Applicant

In FSAR Subsection 2.4S.12.2.2 and its proposed revisions (ML093310392), the applicant describes how the regional deep aquifer flow directions vary over time because of changes in regional and local groundwater withdrawal patterns. In 1967, the groundwater heads were 6.1 to 9.1 m (20 to 30 ft) above MSL in the northern portion of Matagorda County and sloping to sea level at Matagorda Bay. Localized flow disturbances were evident at that time within Matagorda County, which caused an elevated head of more than 12.2 m (40 ft) above MSL and depressions of -33.53 m (-110 ft) below MSL.

While regional potentiometric-level maps are not available for the Shallow Aquifer, local data sets are available from the existing STP site piezometers for STP Units 1 and 2 and the site characterization effort completed for STP Units 3 and 4 (see FSAR Figures 2.4S.12-19 and its proposed revision in ML093310392) showing quarterly data from February 2007 through December 2008). The flow direction in the Upper Shallow Aquifer is described as having components to the east and southeast toward the Colorado River and to the south and southwest along the west side of the main cooling reservoir (ML093310392). In the Lower Shallow Aquifer, the flow direction is described as predominantly toward the east and southeast. The applicant has interpreted the data since September 2008 to indicate that there is no obvious mounding from the main cooling reservoir observed in the Lower Shallow Aquifer (ML092710096).

The recent data indicate that at certain times of the year and at points in the vicinity, there is an upward gradient to Kelly Lake from the Upper Shallow Aquifer, and an upward gradient from the Lower to the Upper Shallow Aquifer is possible (ML083660390). However, at other times of the year and at points in the vicinity of Kelly Lake, the gradients are downward. Thus, there appears to be a seasonal variation (ML102450252), and it is not clear that Kelly Lake is a groundwater discharge location (ML092710096). However, for groundwater flow directions to the east and southeast, the applicant included exposure points at the site boundary, at a private well (i.e., well 2004120846), and at the Colorado River. Although points downgradient of the site boundary to the southeast—including the unnamed tributary feeding Kelly Lake, a private well, Kelly Lake, and the Colorado River—are all plausible, they are conservatively represented by a hypothetical well at the site boundary (ML092710096).

Representative values and ranges of groundwater gradients are taken from the preconstruction potentiometric surfaces for the flow directions considered by the applicant and included in Table 2.4S.12-2 of this SER.

Post-construction ground water simulations show a ground water depression in the vicinity of the power block in the Upper Shallow Aquifer, with releases into that aquifer moving downward into the Lower Shallow Aquifer before migrating to the site boundary (ML102450252 and ML103540324). Field observations at STP Units 1 and 2 of tritium in groundwater and the potentiometric surface confirm this behavior (ML092710096 and ML102450252). Releases into the Lower Shallow Aquifer are projected to move to the east-southeast and to cross the eastern site boundary (ML103540324).

NRC Staff's Technical Evaluation

The staff reviewed FSAR Subsection 2.4S.12.2.2 and its proposed revisions (ML093310392 and ML092710096) and confirmed that the applicant has addressed the relevant information topics.

During a June 2009 review of FSAR and the RAI responses, it was the view of the staff that groundwater flow away from the proposed Units 3 and 4 was plausible to both the southeast and southwest in the Shallow Aquifer. This view was based on site characteristics documented by the applicant that showed groundwater mounding in the Upper Shallow Aquifer and an absence of similar data on the Lower Shallow Aquifer. And in the future, the higher hydraulic head of the Upper Shallow Aquifer will be in direct communication with the Lower Shallow Aquifer, because the excavation within the powerblock will remove the confining strata separating them. The staff received the responses to RAIs (ML092710096, and ML093310392), which included amendments to the FSAR. The staff reviewed the FSAR, the applicant's proposed revisions in these responses, and the revised groundwater model document and found the main cooling reservoir influence and pre-construction (i.e., pre-STP Units 3 and 4) groundwater flow directions are well characterized by the applicant. However, the staff believed the post-construction setting required further evaluation before all plausible future groundwater flow directions could be identified or discarded. Accordingly, in April 2010 the NRC issued supplemental RAIs. The applicant submitted an additional analysis of the post-construction setting (ML102450252). The staff reviewed the RAI responses and noted that the post-construction setting may be well described by three plausible pathways. The applicant provided field data and simulations justifying the exclusion of a west-southwest pathway in the Lower Shallow Aquifer. The FSAR and its revisions include four pathways: (1) the Upper Shallow Aquifer to the east-southeast site boundary, (2) the Upper and Lower Shallow Aquifer to the east-southeast and an existing well, (3) both Shallow Aquifer units to the Colorado River, and (4) a potential Upper Shallow Aquifer discharge to the west-southwest and Little Robin Slough. The staff tracked the applicant's additional sensitivity cases that address other aspects of the Lower Shallow Aquifer pathway to the west-southwest as Open Item 2.4.12-1 in the SER with open items.

Additional sensitivity cases that addressed aspects of the Lower Shallow Aquifer pathway to the west-southwest were submitted by the applicant in a letter dated December 15, 2010 (ML103540324). The staff reviewed the supplemental RAI response (ML103540324) and the ground water documentation (ML110140173). The staff concluded that the Lower Shallow Aquifer flows from the proposed units to the east-southeast site boundary. Based on the site characterization and pre- and post-construction model simulations of the Shallow Aquifer, the staff accepted the applicant's ground waterflow direction. This closes the ground water flow direction aspect of **Open Item 2.4.12-1** in Subsection 2.4S.12.4.12 of this SER.

2.4S.12.4.6 Temporal Groundwater Trends

Information Submitted by Applicant

In FSAR Subsection 2.4S.12.2.3, the applicant presents long-term records for two Deep Aquifer wells monitored by the TWDB that reveal (1) an upper Deep Aquifer that is recovering to 1957 levels (-18.29 m [-60 ft] BGS); and (2) a lower Deep Aquifer, which is the screened interval of the STP wells that is exhibiting a steady groundwater level (-6.1 to -9.1m [-20 to -30 ft] BGS).

The applicant presents water levels within the Upper and Lower Shallow Aquifer for the period from 1994 through 2006. They reveal a high groundwater level in the Upper Shallow Aquifer of approximately 8.23 m (27 ft) MSL adjacent to the site boundary to the east and west of proposed Units 3 and 4. Since early 1997, the variation in the groundwater level of the Upper Shallow Aquifer has been approximately 1.83 m (6 ft). Observation well 929U, completed in the Upper Shallow Aquifer to the northeast of proposed Unit 3, shows a peak groundwater elevation of 8.38 m (27.49) ft MSL. Observation well 993U, between proposed Unit 3 and the main cooling reservoir, shows a peak groundwater elevation of 7.928 m (26.01) ft MSL. Well 602A, completed in the Lower Shallow Aquifer and immediately north of the proposed units, shows a peak groundwater elevation of 7.86 m (25.8 ft) MSL and a variation of approximately 1.22 m (4 ft). Data collected during the STP Units 3 and 4 site characterization efforts reveal between 0.61 and 1.95 m (2.8 and 6.4 ft) of variation from December 2006 through September 2008 in the Upper Shallow Aquifer and between 0.61 and 1.22 m (2.6 and 4.0 ft) of variation in the Lower Shallow Aquifer. Ground water-level data for the Shallow Aquifer show that levels in the Upper Shallow Aquifer are consistently higher than those in the Lower Shallow Aquifer and within approximately 1.5 m (5 ft) BGS during the site characterization period.

During 2007, the Upper Shallow Aquifer piezometric level was steady with a slight decrease after August. The Lower Shallow Aquifer exhibited an increase in the piezometric level until August and then a decrease through December. During 2008, a steadily decreasing trend in piezometric levels was seen in both Shallow Aquifers. This reflected drought conditions in the region.

NRC Staff's Technical Evaluation

The staff reviewed FSAR Subsection 2.4S.12.2.3 and its proposed revisions (ML092710096 and ML093310392) and confirmed that the applicant has addressed the relevant information topics.

The applicant's response to RAI 02.04.12-28 dated August 5, 2009 (ML092170354), which requested the applicant to update the FSAR sections affected by updated data sets with regard to plausible pathways, mounding, gradients, and maps, was completed in the supplemental response dated September 21, 2009 (ML092710096). This RAI response relied, in part, on groundwater model simulations that were revised and submitted later by the applicant. During the review of the FSAR, its proposed revisions, and the groundwater model documentation, the staff concluded that the influence on future mounding in the Lower Shallow Aquifer may have been masked by model bias, and a future hydraulic gradient to the west or southwest may not have been identified. Therefore, a potentially important change in the groundwater system resulting from building the proposed plant may have been omitted. Accordingly, the staff issued RAIs requesting additional information. In a letter dated August 30, 2010, the applicant provided responses to these RAIs (ML102450252).

The staff's review of the applicant's responses clarified the potential for mounding in the Lower Shallow Aquifer, and the potential for a west-southwest directed pathway in the Lower Shallow Aquifer during the post-construction period. The staff identified that field observations of the potentiometric surface in the Upper and Lower Shallow Aquifers in the vicinity of the STP Units 1 and 2 excavation and fill show that removal of the confining sediments between these two aquifers resulted in a groundwater depression in the Upper Shallow Aquifer and a slight groundwater mound in the Lower Shallow Aquifer. These changes occur in the immediate vicinity of the excavation at Units 1 and 2. The staff concludes that a similar response to excavation and fill at the proposed location of Units 3 and 4 can be anticipated. Post-construction simulations of Units 3 and 4 exhibit this behavior; however, the staff found that a west-southwest pathway is not projected to occur in the Lower Shallow Aquifer. Additional sensitivity cases further demonstrated the response to excavation and fill, and the absence of a west-southwest pathway post-construction (ML103540324). This closes the temporal ground water trends aspect of **Open Item 2.4.12-1** described in Subsection 2.4S.12.4.12 of this SER.

The staff reviewed FSAR Subsection 2.4S.12.2.3 and its proposed revisions, and the final sensitivity cases that address aspects of the Lower Shallow Aquifer pathway to the west-southwest (ML103540324) and concluded that the applicant has accurately described the groundwater trends that can be expected at the STP site. Of the trends identified, the staff concur with the normal trend of groundwater rise and decline in response to seasonal change; the trend of declining piezometric levels in response to drought conditions; the anticipated change in the groundwater piezometric levels in response to removing the confining zone materials that separate the Upper and Lower Shallow Aquifers; and the effect of the main cooling reservoir on the Upper Shallow Aquifer.

2.4S.12.4.7 Aquifer Properties

Information Submitted by Applicant

In FSAR Subsection 2.4S.12.2.4, the applicant presented information about precipitation, transmissivity, storativity, hydraulic conductivity, porosity, effective porosity, and bulk density. The applicant also presented the average annual precipitation as 106.7 cm (42 in.) based on a 30-year record from 1951 through 1980 in the vicinity of the STP site. Annual runoff was estimated at approximately 30 cm (12 in.), with the remaining 76.2 cm (30 in.) attributed to the combination of evaporation, transpiration, and infiltration that becomes the recharge to the underlying aquifer. Much of the 76.2 cm (30 in.) is recycled to the atmosphere through evapotranspiration.

The applicant divided the properties of the aquifer between hydrogeological and geotechnical parameters. Transmissivity, storage coefficient, and hydraulic conductivity from tests conducted in the field or derived from such tests are among the hydrogeological parameters. Bulk density (or dry unit weight), porosity, effective porosity, and permeability from grain-sized distributions estimated from laboratory tests are presented as geotechnical parameters.

In FSAR Subsection 2.4S.12.2.4.1, the applicant provides a review of the site-specific measurements and data reductions for hydrogeological parameters against regional parameters shown by Hammond (1969). The applicant concludes that site measurements of Deep Aquifer transmissivity fall within the range of regional values. However, those for the Shallow Aquifer fall below the regional range as a result of a pair of low field measurements in the Upper Shallow Aquifer. The applicant finds that all storage coefficient values fall within the regional range. The applicant compares hydraulic conductivities for the Shallow Aquifer derived from transmissivity measurements and inferred aquifer thickness and slug tests. Hydraulic conductivities for the Shallow Aquifer derived from slug test data were found to fall somewhat below the regional range; however, geometric means of hydraulic conductivity from the two approaches were comparable. Final compilation of the transmissivity and hydraulic conductivity data by the applicant relied on both the FSAR and its proposed revisions (ML09331092). Technical justification for the aquifer pumping test results included in and excluded from the compilation of hydraulic conductivity, transmissivity, and storativity data were provided by the applicant in the supplemental response to RAI 02.04.12-38 (ML102450252).

In FSAR Subsection 2.4S.12.2.4.2, the applicant describes how geotechnical parameters were determined directly or indirectly from laboratory data and reported bulk density as measured in the laboratory. Porosity was calculated using a conversion from void ratio, and effective porosity was estimated as a specific yield using a graphical method relating median grain size to specific yield. Permeability was estimated from grain size using the Hazen approximation, and the applicant found that the geometric mean of hydraulic conductivity values was similar to but lower than that for the STP slug test results. The applicant reports the hydraulic conductivity of clay strata measured during the site characterization effort conducted for STP Units 1 and 2.

The saturated hydraulic conductivity derived from aquifer pumping tests yielded higher geometric means than those derived from slug tests. Therefore, it is the aquifer pumping test results that appear in the applicant's summary table reporting representative properties of the hydrogeologic units (see FSAR Table 2.4S.12-14). Tables 2.4S.12-1 and 2.4S.12-2 of this SER contain a summary of representative property values and their ranges for all geohydrologic strata.

NRC Staff's Technical Evaluation

The staff reviewed FSAR Subsection 2.4S.12.2.4, its proposed revisions (ML093310392), and ground water model documentation. The staff's review confirmed that the applicant has addressed relevant information. The staff also reviewed documents about the hydrology and aquifers of the region from the USGS (Ryder, 1996; Ryder and Ardis 2002), the TWDB (Hammond, 1969; Mace et al., 2006), and the LCRA (Young et al., 2007).

Most of the documents reviewed by the staff report on the Gulf Coast or Coastal Lowlands Aquifer system over a larger region and throughout its depth, especially at inland locations where fresh water has been pumped from deeper strata within the system than are available in Matagorda County. The analysis at the STP site is more local and relies on measurements made locally. However, the staff concluded that data sets of the documented regional models support the aquifer properties found at the STP site, which are summarized in FSAR Table 2.4S.12-14 and shown in Tables 2.4S.12-1 and 2.4S.12-2 of this SER.

The applicant-provided information about aquifer properties in the FSAR resulted in five RAIs to provide consistent interpretations of data. Responses to these RAIs gave rise to additional RAIs including RAI 02.04.12-28, which requested the applicant to incorporate new information into the application. The applicant issued revisions to the FSAR (ML093310392) and a revised groundwater model. The staff's review of the FSAR and its proposed revisions, ER Revision 3, and the revised ground water model led to the identification of inconsistencies in the description of site-specific hydraulic conductivity and transmissivity data. RAI 02.04.12-38 was issued on April 15, 2010 (ML101060021) to gain clarification on these deficiencies. The staff reviewed the applicant's responses (ML101390277 and ML102450252). Based on the applicant's technical justification of the aquifer pumping tests included in and excluded from the data compilation, and the staff's review of the data sets (e.g., hydraulic conductivity, transmissivity, storativity, porosity, bulk density) compared to other studies conducted on the aquifer system, the staff accept the aquifer properties as representative of the Shallow and Deep Aquifer systems underlying the STP site.

2.4S.12.4.8 Hydrogeochemical Characteristics

Information Submitted by Applicant

In FSAR Subsection 2.4S.12.2.5, the applicant provides groundwater quality data for the Deep Aquifer from the mid-1960s, mid-1970s, early 1980s, and one sample from 1991. The applicant also provides data for the Shallow Aquifer from the early 1970s and December 2006. The water quality data are consistent over time and suggest a groundwater system that is not experiencing substantial change. Both aquifers exceed the Secondary Drinking Water Standards (EPA, 2009b) for TDS and chloride. However, there are higher concentrations in the Shallow Aquifer.

The groundwater quality of each aquifer provides a signature that can be used to identify hydraulic connections between aquifers. Within this aquifer system, the Upper Shallow Aquifer is a sodium chloride type while both the Lower Shallow and Deep Aquifers are the sodium bicarbonate type. The Lower Shallow Aquifer exhibits a sodium chloride groundwater type at two onsite locations between the proposed reactors and the Colorado River. This suggests a localized hydraulic connection allowing groundwater from the Upper Shallow Aquifer to enter the Lower Shallow Aquifer. This could be a result of natural (e.g., discontinuous confining unit, incised channel, or scour) or manmade (e.g., pervious backfill, or leaking well) features.

In FSAR Subsection 2.4S.12.2.2, the applicant notes that the hydrogeochemical characteristics of the Shallow Aquifer compared to those of the main cooling reservoir water suggest that there is no strong geochemical correlation between the main cooling reservoir water and the groundwater north of the main cooling reservoir (i.e., in the vicinity of the existing and proposed STP units). In addition, the potentiometric maps indicate little evidence of groundwater mounding north of the main cooling reservoir. This data suggests that relief wells are effective in reducing seepage from the main cooling reservoir to the surrounding groundwater.

NRC Staff's Technical Evaluation

The staff reviewed FSAR Subsection 2.4S.12.2.5, and confirmed that the applicant has addressed relevant information. The staff also reviewed documents containing a discussion of the chemical characteristics of aquifers in the region from the USGS (Ryder and Ardis 2002) and the TWDB (Hammond 1969; Mace et al., 2006). The staff also referred to the EPA Primary and Secondary Drinking Water Standards (EPA 2009a, b). While evaluating and discussing the chemical characteristics on a larger scale, these documents support the applicant's evaluation of hydrogeochemical characteristics in the vicinity of the STP site.

Based on the information in the FSAR, the staff noted that the applicant's description of the hydrogeochemical characteristics of the groundwater resource is an accurate description of groundwater quality in the vicinity of the STP site. Therefore, the staff concluded that the interaction between the Upper and Lower Shallow Aquifers and the main cooling reservoir is a localized phenomenon in the vicinity of the STP site. Therefore RAI 02.04.12-22 is closed.

2.4S.12.4.9 Subsurface Pathways

In FSAR Subsection 2.4S.12.3, and its proposed revisions (ML093310392), the applicant evaluates subsurface pathways to an offsite receptor. Information provided by the applicant includes an evaluation of alternative pathways, an assessment of advective travel times, and results from a model of post-construction ground water flow conditions.

Alternative Pathways Evaluation

Information Submitted by Applicant

In FSAR Subsection 2.4S.12.3.1, the applicant interprets field data to assert that the most plausible ground water pathway for a release from STP Units 3 and 4 is to the east and the southeast in both the Upper and Lower Shallow Aquifer. The applicant also acknowledges a plausible flow component toward the southwest in the Upper Shallow Aquifer.

The excavation for STP Units 3 and 4 penetrates the confining zone separating the Upper and Lower Shallow Aquifer. The hydraulic gradient in the undisturbed system is downward from the Upper to the Lower Shallow Aquifer. Because postulated accidental releases into the Upper Shallow Aquifer in the vicinity of the power block excavation and fill would move downward into the Lower Shallow Aquifer, the applicant concluded that the most likely ground water pathway is the Lower Shallow Aquifer.

Offsite migration pathways for the Upper Shallow Aquifer are to the southeast with the exposure point at (1) the eastern site boundary or an unnamed tributary flowing into Kelly Lake, (2) a private well, and (3) the Colorado River. Kelly Lake is also plausible. The applicant used an exposure point on the site boundary to the east of proposed Unit 3 in the Upper Shallow Aquifer to conservatively represent exposures. The fourth pathway, a southwest pathway in the Upper Shallow Aquifer, is noted to discharge into the headwaters of LRS or into a hypothetical domestic water well installed offsite. The applicant used an exposure point on the site boundary to the west of proposed Unit 4 in the Upper Shallow Aquifer. Offsite migration pathways and exposure points for the Lower Shallow Aquifer are to the east-southeast and are the same as those described for the Upper Shallow Aquifer. The nearest exposure point and, therefore, the most conservative one is a hypothetical domestic water well completed on the eastern site boundary.

The applicant views the Deep Aquifer as the least likely pathway because of the low permeability confining zone separating the Shallow and Deep Aquifers. Releases would likely move to exposure points in the Lower Shallow Aquifer instead of entering and moving through the confining zone. A release that would penetrate the confining zone and enter the Deep Aquifer would be drawn to the production wells, thereby minimizing the potential for offsite migration and exposure. The applicant concluded that there is no credible pathway for offsite exposure involving the Deep Aquifer.

NRC Staff's Technical Evaluation

The staff reviewed FSAR Subsection 2.4S.12.3.1, its proposed revisions, and revised groundwater model documentation. The staff's review confirmed that the applicant has addressed the relevant information topics.

The applicant's response to RAI 02.04.12-17 dated August 12, 2008 (ML082270381), which requested an evaluation of the potential for buoyancy and chelating agents to influence pathways and mobility, was reviewed by the staff and two supplemental RAIs were issued (RAIs 02.04.12-29 and 02.04.12-30) to further evaluate and clarify issues with respect to buoyancy and chelation. While providing a further analysis of the potential for buoyancy to influence pathways, the issue of buoyancy is removed by the inclusion of pathways in the Upper and Lower Shallow Aquifer to the east-southeast and in the Upper Shallow Aquifer to the west-southwest. The applicant adopted the receptor locations on the eastern and western boundaries of the site because they provide the shortest pathways. The applicant showed the receptor location on the eastern boundary of the site in the Upper Shallow Aquifer to be the pathway with the shortest travel time.

Regarding chelating agents, the staff found that the applicant's technical basis is sound for neglecting the potential influences of chelating agents, because it is based on published literature on chelator influences on sediment adsorption, on the expected use and disposal of chelating agents, and on site-specific considerations of substantial dilution by groundwater during any release and an abundance of competing cation clay and silt minerals. RAIs 02.04.12-17, 02.04.12-29, and 02.04.12-30 are considered closed.

The staff reviewed and accepted the applicant's response to RAI 02.04.12-22 dated July 16, 2008 (ML082030326), which requested clarification regarding justification for excluding the Deep Aquifer as a plausible pathway. The applicant states that downward migration from the release point into the Deep Aquifer is not plausible because (1) transport will occur in the media of least resistance (i.e., laterally in the Shallow Aquifer); (2) a 100- to 150-ft-thick confining zone would need to be traversed over a prolonged period of time, and (3) a Deep Aquifer pump test demonstrated the hydraulic isolation of the Deep Aquifer from the Shallow Aquifer. The staff agreed with the applicant's justification to exclude the Deep Aquifer pathway. Therefore RAI 02.04.12-22 is considered closed.

The staff reviewed and accepted the applicant's response to RAI 02.04.12-30 dated September 21, 2008 (ML092710096), which requested information about a release to the ground water environment that could support or refute the conceptual model of downward migration of a liquid radioactive release within the Shallow Aquifer. Tritium concentration data from several Upper and Lower Shallow Aquifer well pairs support the conceptual model of downward migration in the vicinity of STP Units 1 and 2, in response to an unplanned release into the Upper Shallow Aquifer. The concept of a downward migration in the vicinity of the excavation and fill of STP Units 1 and 2 is further substantiated by the applicant's response to RAI 02.04.12-42 dated August 30, 2010 (ML102450252), which provided potentiometric data showing a groundwater depression in the Upper Shallow Aquifer and a ground water mound in the Lower Shallow Aquifer in the vicinity of the existing units. The staff concurred that a downward migration between the Upper and Lower Shallow Aquifers is likely to occur at the proposed location of Units 3 and 4, because construction of the proposed units requires a similar excavation and fill. Therefore RAI 02.04.12-42 is considered closed.

In early 2010, the staff concluded that additional efforts would be required before finalizing the exposure point and pathway evaluation. The staff reviewed the FSAR, its proposed revisions, and the groundwater model documentation. The rationale for the exclusion of a west-southwest pathway in the Lower Shallow Aquifer from proposed Unit 4 was not fully supported. It was apparent that the piezometric head in the Lower Shallow Aquifer could be higher after construction of the plant than measured in the pre-site characterization. The applicant noted in its supplemental response to RAI 02.04.12-28 (ML093310392) that during the site characterization, the west-southwest hydraulic gradient was small and was influenced by seasonal and climatic variability. However, the staff noted that the applicant's interpretation and rejection of a west-southwest pathway from proposed Unit 4 is not based on the possibility of a higher post-construction piezometric head. The staff also noted that the applicant's groundwater model could include a bias that acted to reduce estimates of a future hydraulic gradient to the west or southwest from proposed Unit 4. In addition, the staff noted that the applicant had not evaluated the potential for the permanent, low permeability Crane Foundation Retaining Walls (CFRWs) to influence the groundwater pathways and exposure points. In April 2010 the staff issued supplemental RAIs. Responses were received on August 30, 2010 (ML102450252) and supplemental responses to several RAIs were received on December 15, 2010 (ML103540324).

The applicant's responses to the April 2010 RAIs resulted in a revised and improved preconstruction ground water model (i.e., new topographic data and revised general head boundary [GHB] conditions) and post-construction groundwater simulations based on several updates and design information on structures and structural fill, powerblock finished grade and backfill cover, slurry wall designs, the design of two CFRWs, the relocated MDC, and a conservative representation of the main cooling reservoir water height. Consistent with the tritium and piezometric head observations at STP Units 1 and 2, the post-construction model projects at the proposed Units 3 and 4 site a ground water depression in the Upper Shallow Aquifer and a groundwater mound in the Lower Shallow Aquifer. Despite projecting a ground water mound in the Lower Shallow Aquifer (ML102450252), the simulations did not project a west-southwest pathway from proposed Unit 4. The applicant noted that the model exhibits a bias in the piezometric head toward predicting a southwest pathway (ML102450252). The applicant's supplemental responses (ML103540324) provide an alternative groundwater conceptual model and several sensitivity cases. Based on review of the supplemental responses and groundwater model documentation (ML110140173), the staff accepted the responses and concluded that a west-southwest pathway in the Lower Shallow Aquifer is not plausible. Also, the staff concluded that the most plausible future ground water pathway from the proposed units is in the Lower Shallow Aquifer toward the eastern site boundary.

Therefore, staff found the applicant's description of the alternative ground water pathways acceptable based on the site characterization of the geohydrology of the site and the preconstruction and post-construction ground water model simulations that identify the alternative pathways. This review closes the alternative pathways evaluation aspect of **Open Item 2.4.12-1** in Subsection 2.4S.12.4.12 of this SER.

Advective Travel Times

Information Submitted by Applicant

In FSAR Subsection 2.4S.12.3.2, and its proposed revisions, the applicant provides the analysis of the travel time of ground water along plausible alternative pathways. The applicant's analysis assumes a one-dimensional advective transport of groundwater and associated radioactive contaminants. Such an analysis of contaminant movement assumes that the contaminant moves with the groundwater and is not retarded by geochemical reactions. The average velocity of the pore water in a porous media was estimated using the following equation:

$$v = -(K\ dh/dx)\ /\ n_e$$

where

v = pore water velocity (m [ft]/d)
K = saturated hydraulic conductivity (m [ft]/d)
dh/dx = hydraulic gradient (m [ft]/m [ft])
n_e = effective porosity (decimal)

Travel time (T in days) is then estimated as the distance from the source release to the receptor (D in meter [feet]) divided by the pore-water velocity (v in m [ft]/d). FSAR Table 2.4S.12-17 presents travel times for the four pathways analyzed. The applicant revises and presents this table in a supplemental response to RAI 02.04.12-28 dated November 23, 2009 (ML093310392). The applicant's characterization of the three plausible alternative pathways is shown in Table 2.4S.12-4 of this SER. This table shows the representative value and range for the average linear groundwater velocity and travel time. Estimates of linear ground water velocity use the high estimate of hydraulic gradient derived from the preconstruction piezometric data.

Table 2.4S.12-4. Pathway Average Linear Velocity and Travel Time (from revised FSAR Table 2.4S12-17 [ML093310392]).

		TRAVEL TIME		
Pathway	**Avg Velocity (Range) m/d (ft/d)**	**Representative Value yr**	**Low Range Value yr**	**High Range Value yr**
Upper Shallow to Southeast	0.04 (0.02-0.11) (0.13 [0.05–0.35])	154	57	400
Lower Shallow to Southeast	0.05 (0.03-0.08) (0.16 [0.11–0.26])	125	77	182
Upper Shallow to Southwest	0.02(0.01-0.04) (0.05 [0.02–0.14])	330	117	821

NRC Staff's Technical Evaluation

The staff reviewed FSAR Revision 3, Subsection 2.4S.12.3.2, its proposed revisions, and the revised groundwater model document. The staff's review confirmed that the applicant has addressed the relevant information topics.

In the applicant's supplemental response to RAI 02.04.12-28 (ML092710096), the applicant incorporated updated hydraulic property data into the calculation of advective transport travel times. In the applicant's supplemental response to RAI 02.04.12-38 (ML102450252), the applicant provided technical justification for the hydraulic conductivity data used in their calculations. The staff reviewed these submittals and concurs that the hydraulic conductivity data are justified.

The staff concluded that based on model results and field observations, it is apparent that the hydraulic gradient and travel time estimates for the Upper Shallow Aquifer are conservative because of the likely downward movement of releases from the Upper Shallow Aquifer into the Lower Shallow Aquifer. This closes RAIS 02.04.12-28 and 02.04.12-38 for this subsection.

The applicant's initial estimates of travel time in the Lower Shallow Aquifer were also based on preconstruction hydraulic gradients. Those estimates range from 77 to 182 years with a representative value estimate of 125 years. The applicant provided post-construction simulations and reported a shortest travel time to the site boundary from a postulated Unit 3 release into the Lower Shallow Aquifer of approximately 104 years (see the response to RAI 02.04.12-48 [ML102450252]). Using the site-specific groundwater model (ML110140173), the applicant provides sensitivity cases for the range of saturated hydraulic conductivity and showed a range of travel time from 96 to 127 years (ML103540324). The staff also performed an independent analysis of the influence of a high infiltration rate of 2.54 cm/yr (1 in./yr) and a high backfill hydraulic conductivity of $2.0x10^{-2}$ cm/s (28.35 in./hr), and simulated a post-construction travel time of 94 years. Because the applicant examined both preconstruction and post-construction conditions, and the simulated range of post-construction travel times lies within the range of simple estimates, the staff accepted the applicant's analysis of advective travel times ranging from 77 to 182 years. This review closes the advective travel times aspect of **Open Item 2.4.12-1** in Subsection 2.4S.12.4.12 of this SER.

Groundwater Flow Model

Information Submitted by Applicant

In FSAR Subsection 2.4S.12.3.4, its revision, and the revised ground water model documentation (ML110140173), the applicant describes a three-dimensional, steady-state, numerical groundwater model developed to better understand preconstruction and post-construction ground water conditions at the STP site. The model uses the user interface Visual MODFLOW and is based on the USGS-developed MODFLOW 2000 code. This model was calibrated to the September 2008 hydraulic head data. This data set is among the most complete hydraulic head data sets on the Shallow Aquifer, because it contains data from all site characterization wells completed in the summer of 2008. The applicant applied the calibrated model to simulate post-construction conditions, including excavation and backfill at Units 3 and 4, a slurry wall surrounding Units 3 and 4, and postulated releases from Units 3 and 4. Model results provide projections of groundwater hydraulic head and pathways from releases to receptor points.

In response to a series of RAIs issued in April 2010, the applicant improved the performance of the model by incorporating higher resolution topographic data and adjusting the GHB conditions. These modifications to the model, as revealed in the RAI responses (ML102450252), resulted in an improved match to observed conditions or an explanation for the model behavior (i.e., dry cells were explained, wet cells were substantially reduced, and closed contours in the vicinity of drain boundary conditions were substantially eliminated). Calibration metrics of the improved model are not markedly different from those of the previous model, and the applicant described both the previous and improved models as applicable to the site analysis. Post-construction simulations incorporated design information on the structures and structural fill, power block finished grade and backfill cover, slurry wall designs, design of two CFRWs, relocated MDC, and conservative representation of the main cooling reservoir water height. The post-construction cases confirmed that the most plausible pathway from proposed Units 3 and 4 is to the east-southeast in the Lower Shallow Aquifer. The shortest travel time for the plausible pathway was reported as approximately 104 years. The post-construction model was also used to evaluate the water table elevation within the power block and the potential influence of the relief well system on the maximum groundwater elevation. These simulation results supported the applicant's identified maximum groundwater elevation of 8.5 m (28 ft) MSL.

Supplemental responses (ML103540324) to several of the April 2010 RAIs provided additional sensitivity cases on infiltration rate and backfill saturated hydraulic conductivity, and an improved alternative ground watercalibrated to site conditions.

NRC Staff's Technical Evaluation

The staff reviewed FSAR Subsection 2.4S.12.3.4, its proposed revisions, and the revised groundwater model documentation and simulations.

Responses to the RAIs were received in May and August of 2010, and supplemental responses were received on December 15, 2010. The applicant improved the model by introducing higher resolution topographic data over much of the model and by refining the GHB conditions. The staff understands that these changes involve the model's geometry (i.e., surface and distance to GHB conditions) and the head assigned to the GHB conditions.

Ground water models such as MODFLOW provide the user with a variety of options for implementing a conceptual model of a site. Based on the staff's review of the improved ground water model (ML102450252, ML103540324, ML110140173), the staff concluded that the improved model is suitable as the basis for post-construction simulations, especially with regard to the simulation of the Lower Shallow Aquifer. The staff also reviewed the calibrated model (i.e., the improved model used to simulate a main cooling reservoir at 12.8 m [42 ft] MSL), the improved model applied to a main cooling reservoir at 14.3 m (47 ft) MSL, and the post-construction simulations (main cooling reservoir at 15.1 m [49.5 ft].MSL) The staff concluded that simulation results for the Lower Shallow Aquifer in the region north of the main cooling reservoir would not change substantially by using a further improved model. The simulation of the preconstruction setting exhibits a gradient from the northwest to the southeast that is somewhat higher than observed in the preconstruction Piezometric head data set. The preconstruction and post-construction simulations exhibit piezometric surfaces that support an east-southeastern flow in the Lower Shallow Aquifer and provide an estimate of ground water travel time to the eastern boundary of the STP site. The staff noted that the calibration data set used by the applicant is among the lowest piezometric head data sets available.

The applicant's response to RAI 02.04.12-47 (ML102450252) addresses the issue of boundary conditions and whether they overly constrain post-construction predictions. The applicant examines a number of alternative GHB conditions for the model and performs a number of sensitivity simulations. The applicant concludes that because the variety of GHBs simulated did not result in "any undue impact" on the water table within the power block, the external boundary conditions are acceptable and do not constrain post-construction predictions. Therefore RAI 02.04.12-47 is considered closed.

The applicant response to RAI 02.04.12-48, provides (1) sensitivity simulations of post-construction infiltration rates and hydraulic properties of the backfill; (2) simulations showing the influence of structures, slurry walls, and CFRWs; and (3) simulations showing the failure of the relief well system (ML102450252, ML103540324). The post-construction simulations of both items (2) and (3) (ML102450252) included more detail than the previous model, and the staff concurs that the predicted pathways and groundwater levels are representative of the conditions simulated. Simulation of item (1) (ML103540324) included several sensitivity cases including a range of infiltration rates and backfill hydraulic conductivities. The staff concurs that the groundwater model is representative of site conditions and the power block region, and it is sufficient to evaluate groundwater elevation and plausible pathways in response to pre- and post-construction site conditions (e.g., changes in grade, structures, and increased main cooling reservoir level). This review closes the groundwater flow model aspect of RAI 02.04.12-28 and **Open Item 2.4.12-1** in Subsection 2.4S.12.4.12 of this SER.

2.4S.12.4.10 Monitoring or Safeguard Requirements

Information Submitted by Applicant

In FSAR Subsection 2.4S.12.4, the applicant describes monitoring of the groundwater system from preconstruction through the startup of the plant. In the ER in Revision 3, the applicant states that during the preconstruction period, ground water levels will be monitored at up to 54 wells that provide observations in both the Upper and Lower Shallow Aquifers. During the detailed design of proposed STP Units 3 and 4, the applicant will review current STP ground water monitoring programs to identify necessary modifications to incorporate into the monitoring of the proposed units. The review will consider the needed water-level and water-quality measurements for the Deep and Shallow Aquifers, subsidence monitoring in the vicinity of proposed Units 3 and 4, and operational accident monitoring. The applicant will use the reviewed and modified ground water monitoring programs to monitor ground water in the Deep and Shallow Aquifers during the construction and preoperational monitoring periods. The applicant will use groundwater monitoring during construction to track changes in groundwater resulting from construction activities including the slurry cut-off wall, CFRWs, and excavation dewatering.

In the ER, the applicant is committed to use best management practices, including well-head protection, to protect the aquifers. The applicant anticipates that the ground water monitoring required during the operation of proposed Units 3 and 4 will be similar to existing reporting requirements for STP Units 1 and 2 and will be designed and implemented accordingly. However, the applicant acknowledged that the requirements are changing in response to the Nuclear Energy Institute's program to collect groundwater data at commercial nuclear plants. Once construction is complete and the sediment profile has been allowed to rewet, the applicant has committed to the continued evaluation of groundwater levels with the objective of determining whether groundwater level monitoring should continue to ensure that the maximum groundwater level beneath safety-related structures of proposed STP Units 3 and 4 is greater than 61 cm (2 ft) below plant grade at all times (ML082100162).

NRC Staff's Technical Evaluation

The staff reviewed FSAR Subsection 2.4S.12.2.4, and confirmed that the applicant has addressed relevant information. The staff also reviewed ER sections that address groundwater monitoring and radiological environmental operating reports for STP Units 1 and 2 in 2006 and 2007 (Sherwood and Travis 2007, 2008).

In the FSAR, the applicant describes monitoring and safeguard requirements by stating that current STP monitoring will be evaluated to determine whether any modifications of existing programs are required to adequately monitor the proposed units. Considerations include the following monitoring:

- deep aquifer monitoring of hydraulic head and geochemical quality to detect influences on the groundwater supply or accidental releases
- shallow aquifer monitoring of hydraulic head and geochemical quality to detect changes in flow patterns, potential changes to accident analysis, potential influences on structural stability, and structural integrity
- subsidence monitoring to ensure structural stability
- operational accident monitoring to detect the presence of radionuclides in the environment

The staff reviewed and accepted the applicant's responses to RAI 02.04.12-24 dated July 24, 2008 (ML082100162), which requested clarifications regarding the ground water monitoring program and its objectives. The applicant's RAI response describes the current monitoring program and commits to review and modify the current plan to address the proposed units. In addition, the RAI response describes the expansion of the current monitoring plan to incorporate industry guidelines for the detection and monitoring of releases of plant-related radionuclides into the ground water environment. The staff concluded that the applicant's description of how STP will meet the monitoring requirements is appropriate. Therefore RA 02.04.12-24 is considered closed.

The staff noted that the applicant's response to RAI 02.04.12-24 states that STP will perform ground water-level monitoring "during construction dewatering and rewetting activities" and will evaluate ground water-level observations after the profile has rewetted to determine whether continued monitoring is warranted or not.

2.4S.12.4.11 Site Characteristics for Subsurface Hydrostatic Loading

Information Submitted by Applicant

In FSAR Subsection 2.4S.12.5, and its proposed revisions, the applicant summarizes the evaluation of hydrostatic loading estimates based on the plant grade and the site characteristic maximum groundwater level. The applicant provides changes to FSAR Subsection 2.4S.12.5 in response to RAI 02.04.12-35 dated September 21, 2009 (ML092710096) and comments on support of the site characteristic in response to RAI 02.04.12-34 (ML092710096) and RAI 02.04.12-49 (ML102450252, ML110450097).

The applicant adopts a site characteristic for a maximum groundwater level of 8.5 m (28 ft) MSL based on field measurements and modeled post-construction results. The applicant states that the post-construction plant grade will be approximately 10.4 m (34 ft) MSL. According to the DCD requirement (i.e., maximum ground water level is to be greater than 61 cm [2 ft] BGS), the maximum ground water level shall be no higher than 9.75 m (32 ft) MSL. The applicant evaluates hydrostatic loading by comparing two calculations of hydrostatic load that are (1) based on the DCD requirement, and (2) based on the site characteristic. The applicant states that the site characteristic of 8.5 m (28 ft) MSL satisfies the DCD requirement of 61 cm (2 ft) below plant grade and exhibits a satisfactory hydrostatic pressure.

Support for the selection of a site characteristic of 8.5 m (28 ft) above MSL lies in the field observations of preconstruction groundwater levels inside the power block (ML110450097):

- Over a 34-year period from 1973 through 2007, groundwater levels were below 8.38 m (27.5 ft) MSL in the northern portion of the STP site (ML081890239 and ML082100162).

- Piezometer 602A, the piezometer located nearest to the proposed units, during 1995 through 2006 recorded groundwater elevations below 7.93 m (26 ft) MSL (ML092710096).

- The observation wells within the footprint of the proposed units during 2007 and 2008 show a maximum groundwater elevation of 7.91 m (25.94 ft) MSL (ML092710096).

Support for the selection also lies in the results of post-construction groundwater simulations (ML102450252, ML103540324, ML110140173, ML110450097):

- Post-construction scenarios simulated with the slurry wall in place showed post-construction groundwater levels 30 to 91 cm (1 to 3 ft) lower than preconstruction levels in the Upper Shallow Aquifer in the vicinity of safety-related facilities for Units 3 and 4.

- The post-construction scenarios (including the slurry wall and with the main cooling reservoir at 15.1 m [49.5 ft] MSL) simulated a maximum groundwater level within the proposed power block of about 6.4 m (21 ft) MSL (see Figure 62 in ML110140173).

Additional support for the selection comes from field observations at STP Units 1 and 2 that confirm the creation of a ground water depression in the region excavated and backfilled. A 0.91- to 1.52-m (3- to 5-ft) depression in the piezometric surface is seen in a May 2006 data set (ML102450252). These observations support the post-construction simulation of the ground water depressions at STP Units 1 and 2 and proposed Units 3 and 4. A sensitivity case simulated to learn the relationship between the relief wells surrounding the main cooling reservoir examined the case of all relief wells hypothetically removed, and the ground water elevation within the power block showed a maximum groundwater elevation of approximately 7.86 m (25.8 ft) MSL at the south side of the slurry wall and a simulated maximum groundwater level of less than 7.62 m (25 ft) MSL within the power block.

The applicant concludes that based on historical evidence and post-construction groundwater model results, the maximum post-construction groundwater level at the proposed STP Units 3 and 4 of 8.5 m (28 ft) MSL will not be exceeded, and this site characteristic meets the DCD requirement for the maximum ground water level.

Based on several factors, the applicant also concludes that "a permanent dewatering system is not anticipated to be a design feature of STP Units 3 & 4." These factors include the following:

- The site characteristic of a maximum groundwater level of 8.5 m (28 ft) MSL.

- Most of the power block surface will be occupied by buildings, structures, and relatively low permeability material (asphalt, concrete). With the exception of buildings and their foundations, the entire power block will be underlain by a low permeability clay layer a minimum of 2 ft thick. Such a power block surface and subsurface will minimize the potential for infiltration and recharge.

- Observations of the STP Units 1 and 2 post-construction water table compared to the pre-construction water table.

With regard to the post-construction power block, roof drains will flow to storm drains. The surface grade within the power block will direct runoff from low-permeability surfaces to storm drains. Storm drains will direct stormwater away from the power block and discharge into surface-water outfalls. With regard to post-construction observations, the effects on the water table from the construction and operation of STP Units 1 and 2 suggest localized changes in the hydraulic head, including communication between the Upper and Lower Shallow Aquifers and an increased drawdown in the Deep Aquifer resulting from production well pumping. The applicant's response to RAI 02.04.12-26 presented a 34-year record of water-level data for the Upper Shallow Aquifer in the vicinity of STP Units 1 and 2. The applicant concludes that ground water elevations measured before construction of STP Units 1 and 2 have been a good indicator of groundwater elevations after the construction of these units. The applicant assumes that this same concept will apply to STP Units 3 and 4.

NRC Staff's Technical Evaluation

The staff reviewed FSAR Subsection 2.4S.12.2.5 and its proposed revisions and confirmed that the applicant has addressed relevant information.

Based on the results of the probabilistic seismic hazard analysis discussed in FSAR, Section 2.5.2, Vibratory Ground Motion, the staff determined that it is unlikely that ground motion level or associated liquefaction could affect the maximum ground water level at the STP site. Therefore, a detailed evaluation of this seismically induced ground water table arising was not performed by the staff as part of this section.

As discussed in FSAR Section 3.8, the DCD site parameter "maximum groundwater level" has been used in conjunction with the most required load combinations, including normal loads and the combination with earthquake loads, the combination with severe winds, and the combination with tornado loads. The DCD site parameter "maximum flood level" is exceeded at the STP site during the design basis flood event, and the maximum groundwater conditions associated with this event are included in the engineering evaluation. The staff's review for subsurface hydrostatic loading is divided into two review topics: (1) the maximum ground water level under normal conditions and all extreme events excluding the "maximum flood level," and (2) the maximum ground water level during the event resulting in the "maximum flood level."

With regard to the first topic, the staff independently assessed the maximum groundwater elevation during the post-construction period. The staff obtained an estimate of the maximum groundwater elevation by adding the maximum observed variation in piezometric head (i.e., 1.44 m [4.71 ft]) to the simulated post-construction ground water within the power block (i.e., 6.4 m [21 ft] MSL). This approach yields an estimate for a maximum ground water elevation of 7.84 m (25.71 ft) MSL. A second and similar estimate of maximum ground water elevation is obtained by adding the delta in the model simulation (i.e., the difference between post-construction and preconstruction; negative 0.3 to 0.9 m (1 to 3 ft) implying a ground water depression), to the observed maximum preconstruction ground water piezometric head (i.e., 7.91 m [25.94 ft] MSL). This approach yields an estimate for maximum ground water elevation of approximately 7.62 m (25 ft) MSL. Neither estimate exceeds the proposed site characteristic of 8.5 m (28 ft) MSL.

The staff's review of the site characteristic for the maximum groundwater level also includes consideration of events other than the design-basis flood resulting in surface water inundating the site, (e.g., storm surge, tsunami, dam breach, river flooding, or precipitation conditions resulting in minor flooding). The staff determined that the mechanism that would result in the condition associated with a maximum ground water level could be any of the above example events. Using soil physics theory to estimate the movement of the wetting front (Jury et al., 1991), the staff estimated that the wetting front would require 28 days to penetrate the 0.6-m-thick (2-ft-thick) clay cap, and years to saturate the upper portion of the natural clay deposit overlying the STP site. The periods of time required for the wetting front to penetrate the clay materials exceeds the duration of any flood event, assuming the clay layers remain intact. Because none of the events would result in scour of the surface profile, substantial areas of the engineered backfill would not be exposed to the surface water. However, in response to RAI 02.04.12-51 (ML103330369), the applicant notes that minor excavations into the clay cap could occur over the life of the plant. The applicant also notes the large extent of the aquifer and the limited extent of future excavations through the clay cap, and concluded that the amount of infiltration would not affect the groundwater level. Although this is dependent on the extent of future excavation, the staff concurred that infiltration into the engineered backfill would be local to such excavations and limited with regard to influence on the overall ground water level within the power block area.

With regard to long-term precipitation and infiltration the applicant completed a sensitivity simulation (ML103540324) that involved a high estimate of long-term infiltration (2.54 cm/yr or 1 in./yr), and the low estimate of engineered backfill saturated hydraulic conductivity (5.0×10^{-4} cm/s [10.6 gpd/ft^2]). This sensitivity simulation was designed to determine the probable maximum groundwater level in the power block area. This simulation resulted in a predicted piezometric head in the Upper Shallow Aquifer well below the site characteristic for maximum groundwater level of 8.5 m (28 ft) MSL. The staff re-simulated and reviewed this case and confirmed the applicant's conclusion.

With regard to the first topic, the maximum groundwater level under normal conditions and all extreme events excluding the "maximum flood level," the staff reviewed the applicant's submittals and performed independent calculations that confirm the applicant's defined site characteristic for maximum groundwater level at 8.5 m (28 ft) MSL.

With regard to the second topic, the maximum groundwater level during the event resulting in the "maximum flood level," the staff's review focused on the design-basis flood, which is the main cooling reservoir breach and flood analysis (see Section 2.4S.4 of this SER). In Section 2.4S.4 of this SER, the staff confirmed that the design-basis flood of 12.2 m (40 ft) MSL is not exceeded. Also in Section 2.4S.4 of this SER, the staff assumed conservatively that the clay cap could be eroded away during the design-basis flood. The erosion of the clay cap would expose the engineered backfill to surface waters for the duration of the design-basis flood. Using soil physics theory (Jury et al., 1991) to estimate wetting front movement under surface water ponded conditions, the staff estimates that infiltration into the engineered backfill would result in saturation of the entire vertical profile from the plant grade to the level of 8.5m (28 ft)

MSL. This groundwater conditions is included in the engineering evaluation as discussed in SER Section 3.8.

The staff reviewed FSAR Subsection 2.4S.12.2.5, its proposed revisions, and the applicant's response to RAI 02.04.12-51 and confirmed that the applicant has provided the staff with sufficient information and analyses to close RAI 02.04.12-51 and, with regard to Open Item 2.4.4-1 to close Open Item 2.4.12-1. RAI 02.04.12-51 was issued to obtain information and analyses regarding infiltration during the design-basis flood event. The staff determination resulting in separate reviews of the site characteristic for "maximum groundwater level" and groundwater conditions during the design-basis flood enabled staff to complete the review with the information and analyses provided.

The applicant completed the sensitivity cases described in the August 2010 submittal and submitted a summary of the results on December 15, 2010 (ML103540324). These results were supplemental responses to RAIs **02.04.12-46, 02.04.12-48**, and **02.04.12-50**, and provided further technical justification for the post-construction subsurface pathways and groundwater level. The applicant submitted the revised ground water model documentation and the ground water model input and output files on January 11, 2011. Based on staff's review of applicant's submittals (ML103540324, ML110140173) described above, the responses to these RAIs and the groundwater model are accepted, and this portion of **Open Item 2.4.12-1** is closed.

The applicant describes and estimates the potential for the design-basis flood to cause infiltration through the surface and affect (1) the ground water level within the power block (i.e., could the water table approach or exceed the site characteristic); and (2) the saturation of the upper 2 ft of sediment (i.e., could the subsurface between the plant grade and 2 ft below plant grade become saturated). The applicant considered flood scour and erosion of the power block surfaces and will maintain the surfaces. Based on the staff's review of the applicant's submittals (ML103330369 and ML103630545) with regard to **RAI 02.04.12-51** described above, the responses to these RAIs are accepted and this portion of **Open Item 2.4.12-1** is closed. RAI 02.04.12-49 and 02.04.12-51 will be tracked as Confirmatory Item 02.04.12-2.

The applicant's RAI responses and associated FSAR revisions demonstrated the strong technical basis for the plausible alternative pathways and their simulation, the site characteristic of the maximum ground water level, and that the design bases related to ground water-induced loadings on subsurface portions of safety-related SSCs would not be exceeded under normal conditions and all extreme events excluding the maximum flood level. Accordingly, **Open Item 2.4.12-1** is closed.

2.4S.12.5 Post Combined License Activities

There are no post-COL activities related to this subsection.

2.4S.12.6 Conclusion

The staff reviewed the application and confirmed that the applicant has addressed the required information related to site groundwater characteristics. The staff confirmed that the applicant has included complete descriptions of the current and future local hydrological conditions, including alternate conceptual models, to demonstrate that the design bases related to groundwater-induced loadings on subsurface portions of safety-related SSCs would not be exceeded. The staff accepted the methodologies used to determine the potential effects of ground water as documented in safety evaluation reports.

As set forth above, the applicant has presented and substantiated information relative to the ground water effects important to the design and siting of the proposed plant. The staff reviewed the available information provided and, for the reasons given above, concluded that the identification and consideration of the potential effects of ground water in the vicinity of the site are acceptable and meet the requirements of 10 CFR 52.79, 10 CFR 100.20(c)(3), 10 CFR 100.23(d)(3), and 10 CFR 100.20(c), with respect to determining the acceptability of the site. The relevant information addressing COL License Information Item 2.32, (i.e., "that the COL applicant analyze the groundwater condition for the specific site"), is adequate and acceptable.

2.4S.12.7 References

Coastal Plains Groundwater Conservation District (CPGCD) 2009. Email from Neil Hudgins to Charles Kincaid dated April 20, 2009; subject: RE: Request for information on "desired future conditions" and "managed available groundwater." Accession No. ML100670653.

Hammond, Jr., W.W., 1969. Ground-Water Resources of Matagorda County, Texas. Texas Water Development Board Report 91. Texas Water Development Board, Austin, Texas.

Jury, W.A., W.R. Gardner, and W.H. Gardner. 1991. Soil Physics, fifth edition. John Wiley & Sons, Inc., New York, pages 128-137.

Lower Colorado Regional Water Planning Group, 2006. Region "K" Water Plan for the Lower Colorado Regional Water Planning Group. Available online at http://www.twdb.state.tx.us/rwpg/2006_RWP/RegionK/

Lower Colorado Regional Water Planning Group, 2009. "LCRWPG 2011 Water Plan First Biennium Studies; Surface Water Availability Modeling Study for the Lower Colorado Regional Water Planning Group." Available online at http://www.twdb.state.tx.us/RWPG/rpgm_rpts/0704830696Surface_Water_Availability_Modeling_Study1.pdf

Mace, R.E., S.C. Davidson, E.S. Angle, and W.F. Mullican, III, 2006. "Aquifers of the Gulf Coast of Texas," Texas Water Development Board Report 365. Texas Water Development Board, Austin, Texas.

Reynolds, K.W. 2007. "2006 Radioactive Effluent Release Report," South Texas Project Electric Generating Station, Bay City, Texas.

Ryder, P.D. 1996. Groundwater Atlas of the United States, Oklahoma, Texas. HA 730-E. U.S. Geological Survey. http://pubs.usgs.gov/ha/ha730/ch_e/index.html.

Ryder, P.D. and A.F. Ardis, 2002. Hydrology of the Texas Gulf Coast Aquifer Systems; Regional Aquifer-System Analysis – Gulf Coastal Plain. U.S. Geological Survey Professional Paper 1416-E. U.S. Geological Survey, Denver, Colorado.

Sherwood, J.D. and P.L. Travis, 2008. 2007 Annual Environmental Operating Report South Texas Project Electric Generating Station. STP Nuclear Operating Company, Bay City, Texas.

Sherwood, J.D. and P.L. Travis, 2007. 2006 Annual Environmental Operating Report South Texas Project Electric Generating Station. STP Nuclear Operating Company, Bay City, Texas.

STPEGS. 2006. Updated Final Safety Analysis Report for Units 1 and 2, Revision 13, May 2006. STPEGS. Bay City, Texas.

Texas Water Code. Chapter 36 Groundwater Conservation Districts. Accessed April 16, 2009 at http://tlo2.tlc.state.tx.us/statutes/wa.toc.htm
to obtain copy of Chapter 36 at
http://tlo2.tlc.state.tx.us/statutes/docs/WA/content/pdf/wa.002.00.000036.00 .pdf

Texas Water Development Board. 2006. 2007 State Water Plan; Water for Texas 2007. available online at http://www.twdb.state.tx.us/wrpi/swp/swp.htm

Turner, Collie, and Braden, Inc. 2004. Groundwater Management Plan prepared for: Coastal Plains Groundwater Conservation District. Coastal Plains Groundwater Conservation District, Bay City, Texas.

U.S. Environmental Protection Agency (EPA). 2009a. Drinking Water Standards http://www.epa.gov/ogwdw/standars.html accessed 6/8/2009 1:09:16 PM

U.S. Environmental Protection Agency (EPA). 2009b. Secondary Drinking Water Regulations: Guidance for Nuisance Chemicals; http://www.epa.gov/OGWDW/consumer/2ndstandards.html accessed 6/8/2009 11:14:26 AM)

U.S. Nuclear Regulatory Commission. 1975. Final Environmental Statement related to the proposed South Texas Project Units 1 and 2. NUREG-75/019. U.S. Nuclear Regulatory Commission, Office of Nuclear Reactor Regulation, Rockville, Maryland.

U.S. Nuclear Regulatory Commission. 1986. Final Environmental Statement related to the operation of South Texas Project, Units 1 and 2. NUREG-1171. U.S. Nuclear Regulatory Commission, Office of Nuclear Reactor Regulation, Rockville, Maryland.

Young, S.C., V. Kelley, T. Budge, and N. Deeds. 2007. Final; Development of the LSWP Groundwater Flow Model for the Chicot and Evangeline Aquifers in Colorado, Wharton, and Matagorda Counties.

2.4S.13 Accidental Releases of Radioactive Liquid Effluent in Ground and Surface Waters

2.4S.13.1 Introduction

This section of the FSAR considers the potential effects of accidental releases from the radwaste management systems that handle liquid effluents generated during normal plant operations. Such releases would have relatively low levels of radioactivity, but they could be large in volume. Normal and severe accidental releases are also considered in the applicant's ER and FSAR Chapter 15. The accidental release of radioactive liquid effluents in ground and surface waters is evaluated based on the hydrogeological characteristics of the site. The source term from a postulated accidental release is reviewed under SRP Section 11.2 following the guidance in Branch Technical Position (BTP) 11-6, "Postulated Radioactive Releases Due to Liquid-containing Tank Failures" (NRC 2007). The source term is determined from a postulated release from a single tank outside of the containment.

This SER section provides an evaluation of the ability of the ground water and surface-water environment to delay, disperse, dilute, or concentrate liquid effluent, as related to existing or potential future water users.

2.4S.13.2 Summary of Application

In Section 2.4S.13, the applicant addresses the accidental release of radioactive liquid effluents in ground and surface waters. In addition, the applicant provides a site-specific supplement designed to address COL License Information Item 2.21.

<u>*COL Information Item*</u>

- COL Information Item 2.21 — Accidental Release of Liquid Effluents in Ground and Surface Waters"

COL license information item directs the applicant to provide site-specific information to address the accidental release of radioactive liquid effluents in ground and surface waters by (1) providing information about the ability of the surface- and subsurface-water environment to disperse, dilute, or concentrate accidental releases; and (2) describing the effects of these releases on existing and known future uses of water resources.

2.4S.13.3 Regulatory Basis

The associated acceptance criteria are in Section 2.4.13 of NUREG–0800.

The applicable regulatory requirements for liquid effluent pathways for groundwater and surface water are as follows:

- 10 CFR Part 100, as it relates to identifying and evaluating hydrological features of the site. The requirement to consider physical site characteristics in site evaluations is specified in 10 CFR 100.20(c).

- 10 CFR 100.23(d)(3), as it sets forth the criteria to determine the siting factors for plant design bases with respect to seismically induced floods and water waves at the site

- 10 CFR 20, as it relates to effluent concentration limits

- 10 CFR 52.79(a)(1)(iii), as it relates to identifying hydrologic site characteristics, with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and the surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

The regulatory positions of the following documents are used for the related acceptance criteria:

- BTP 11-6, "Postulated Radioactive Releases Due to Liquid Containing Tank Failures," provides guidance in assessing a potential release of radioactive liquids after the postulated failure of a tank and its components located outside of the containment, and the impacts of the release of radioactive materials at the nearest potable water supply located in an unrestricted area for direct human consumption or indirectly through animals, crops, and food processing.

- RG 1.113, "Estimating Aquatic Dispersions of Effluents from Accidental and Routine Reactor Releases for the Purpose of Implementing 10 CFR Part 20, Appendix B effluent concentration limits.

2.4S.13.4 Technical Evaluation

The staff reviewed the information in Section 2.4S.13 of the STP Units 3 and 4 COL FSAR. The staff's review confirmed that the information in the application addresses the relevant information related to the accidental release of radioactive liquid effluents in groundwater and surface water. The staff's technical review of this section includes an independent review of the applicant's information in the FSAR and in the responses to RAIs. The staff supplemented this information with other publicly available sources of data.

This section describes the staff's evaluation of the technical information presented by the applicant in FSAR Section 2.4S.13.

COL License Information Item

- COL License Information Item 2.21 — Accidental Release of Liquid Effluents in Ground and Surface Waters"

Specific information provided by the applicant to address COL Information Item 2.21 includes all material presented in FSAR Section 2.4S.13. The staff reviewed the applicant's FSAR, its revision, and RAI responses with regard to the accidental release of liquid effluent in ground water and surface water. The staff's review of the application is summarized in the following subsection. The staff reviewed the applicant's submittals using RG 1.206 and the review procedures described in Section 2.4.13 of NUREG–0800.

2.4S.13.4.1 Direct Release to Groundwater

Information Submitted by Applicant

In FSAR Subsection 2.4S.13.1, and its proposed revisions, the applicant provides an analysis of the postulated accidental liquid release into groundwater at the STP site. The applicant includes in the pathway analysis the processes of advection, dispersion, retardation, and decay. The applicant analyzes the plausible alternative pathways developed and presented in FSAR Section 2.4S.12. The analysis was applied to the plausible alternative pathways in two stages. The first stage considers non-retarded groundwater travel time (advection) and decay only to eliminate the majority of radionuclides that have relatively short half-lives. The second stage considers advection, dispersion, retardation, and decay to evaluate all radionuclides that pass the first stage. Reactions with the sediments can reduce the radionuclide concentration through cation/anion exchange, complexation, oxidation-reduction, and surface sorption. The applicant chose to simulate the combination of geochemical reactions with the linear sorption isotherm model.

NRC Staff's Technical Evaluation.

The staff reviewed the introductory material and FSAR Subsection 2.4S.13.1. The staff's review confirmed that the applicant has addressed relevant information. The applicant quoted from ABWR DCD Tier 2, Subsection 15.7.3.3, and the staff confirmed the following statement regarding a postulated radioactive release due to liquid radwaste tank failure: "(t)he liquid pathway is not considered due to the mitigative capabilities of the Radwaste Building." Furthermore, the staff noted in ABWR DCD Tier 2, Subsection 12.2.1.2.10, the following statement: "potential releases in the Radwaste Building will be contained by filtering the Radwaste Building atmosphere and sealing any water releases in the building, which is steel-lined to prevent any potential water releases." The applicant quotes from ABWR DCD Tier 2, Subsection 15.7.3.1, and NRC confirmed that "(t)he probability of a complete tank release is considered low enough to warrant this event as a limiting fault." However, for the purpose of conservatism, the applicant concludes and NRC confirmed that the postulated rupture of a radwaste tank in the ABWR radwaste building is considered limiting for the analysis of accidental releases of radioactive liquid effluents in ground water and surface water.

The staff accepted the applicant's statements describing the ground water pathway as being conservatively represented by the processes of advection, dispersion, retardation, and decay. The staff also accepted that geochemical reactions between the radioactive liquid effluent and the aquifer matrix could include cation/anion exchange, complexation, oxidation-reduction reactions, and adsorption on surfaces. And the staff acknowledged that decay can be significant, especially for short-lived radionuclides. The staff concluded that the applicant's general description of the direct release into ground water is accurate.

The staff reviewed the statements of this section and the description of the approach in the response to RAI 02.04.13-1, which requested a description of the process followed to identify plausible alternative pathways. The latter was incorporated by the applicant into FSAR Subsection 2.4S.12.1.1. The staff found the statements of the process of data review and assimilation to formulate plausible alternative pathways and conceptual models satisfactory. Therefore, RAI 02.04.13-1 is considered closed.

2.4S.13.4.2 Accident Scenario

Information Submitted by Applicant

In FSAR Subsection 2.4S.13.1.1, the applicant-postulates that the release into ground water and surface water is from a liquid radwaste tank rupture in the radwaste building. The applicant's review of radioactive sources concludes that the low conductivity waste (LCW) collector tank would be the best choice based on its having the greatest concentrations of radioisotopes. There are four LCW collection tanks, each with a volume of 140 m^3 (36,984 gal) (ABWR DCD Tier 2, Section 11.2, Table 11.2-4). Based on BTP 11-6 guidance, the postulated rupture of one LCW collector tank is assumed to release 80 percent of its liquid volume (112 m^3 [29,587 gal]) into the ground water environment. The release into the ground water is assumed to reach the aquifer without being diluted. The radionuclide concentrations assigned to the tank rupture in FSAR Table 2.4S.13-1, are the highest radionuclide concentrations from either the LCW collector tank concentrations or the reactor coolant concentrations (DCD Tier 2, ABWR Revision 4, Section 11.1).

The applicant notes that the radwaste building includes numerous components that make a release into groundwater from a radwaste tank in the building unlikely. The building design includes a basemat and walls to a height needed to retain spilled liquids, and the rooms containing the LCW tanks are steel lined to a height capable of retaining the contents of the tank. Furthermore, the rooms are equipped with alarmed tank-level monitoring and a sump collection system to collect any leakage. Part 7 of the Departures Report STD DEP 11.2-1 states that a release into the groundwater is not considered credible.

NRC Staff's Technical Evaluation

The staff reviewed FSAR Subsection 2.4S.13.1.1. The staff's review confirmed that the applicant has addressed relevant information. The staff reviewed the postulated release into ground water and surface water and found the postulated liquid radwaste tank rupture in the radwaste building to be consistent with the ABWR DCD information and BTP 11–6. The staff reviewed and accepted the radionuclide concentrations reported in FSAR Table 2.4S.13-1, as the highest from either the LCW collector tank or the reactor coolant concentrations (DCD Tier 2, Revision 4, Section 11.1).

2.4S.13.4.3 Conceptual Model

Information Submitted by Applicant

In FSAR Subsection 2.4S.13.1.2, and its proposed revisions, the applicant describes the conceptual model(s) used to evaluate the plausible ground water pathways that an accidental release of radioactive liquid effluent could follow at the proposed STP site. The model is based on the hydrogeological data and interpretations in FSAR Section 2.4S.12. In brief, there are two aquifers underlying the STP site: the Shallow Aquifer and the Deep Aquifer. They are separated by a 30 to 46 m (100- to 150-ft) thick deposit of clay and silt. The Shallow Aquifer can be subdivided into an Upper and Lower Shallow Aquifer that are separated by an approximately 6.1 m (20 ft) thick deposit of clay and silt.

In FSAR Section 2.4S.12, the applicant concludes that groundwater flow is predominantly to the east-southeast from the reactor location toward the Colorado River in the Upper and Lower

Shallow Aquifer, with some potential for flow toward the southwest and the western side of the STP site in the Upper Shallow Aquifer. Along the predominant pathway to the southeast, the applicant selected three exposure points as plausible: (1) the east site boundary where the Upper Shallow Aquifer could be intercepted by an unnamed tributary and where the Upper and Lower Shallow Aquifer could release into a hypothetical offsite water-supply well; (2) an existing offsite water well (number 2004120846); and (3) the Colorado River. Kelly Lake is also identified as a plausible receptor location. All east and southeast directed pathways, including Kelly Lake, are conservatively represented by a pathway intercepted by a hypothetical water-supply well assumed to be located at the eastern site boundary. The applicant also admitted a fourth pathway, a west-southwest pathway in the Upper Shallow Aquifer from proposed Unit 4 to the western site boundary, where a similar hypothetical water-supply well is assumed to be located.

Based on the site characterization data showing that the hydraulic head of the Upper Shallow Aquifer is higher than that in the Lower Shallow Aquifer, the applicant has concluded that a downward hydraulic gradient will likely force radioactive liquid effluents released into the Upper Shallow Aquifer downward into the Lower Shallow Aquifer. The applicant also noted that the third exposure point to the southeast, which releases into the Colorado River, would represent a combined groundwater and surface-water pathway that could be further analyzed using the minimum 7-day low-flow rate of the Colorado River, approximately 14.2 Lps (0.5 cfs). This low-flow value is based on Colorado River flow data from 1948 through 2006.

The applicant considers and eliminates several exposure points, pathways, or transport processes (i.e., the applicant found them not to be plausible), as follows:

- exposure at or attributed to the relief wells surrounding the main cooling reservoir
- exposure from a Deep Aquifer pathway
- exposure at the western side of the STP site from a southwest groundwater pathway in the Lower Shallow Aquifer
- a pathway in the Upper Shallow Aquifer related to the thermal buoyancy during the release
- enhanced transport because of the presence of chelating agents

The applicant finds the exposure points, pathways, or transport processes described above not to be plausible for the following reasons, respectively:

- Ground water flow is from the main cooling reservoir past the relief wells and toward STP Units 3 and 4. Therefore, relief wells will not be exposure points.
- The Deep Aquifer is separated from the Shallow Aquifer by a low-conductivity confining unit of at least 30 m (100 ft) of clay and silt, and piezometric level data show that groundwater in the Deep Aquifer underlying the STP site is drawn to the STP production wells, making an offsite release unlikely.
- The applicant concludes from potentiometric and hydraulic conductivity data that ground water flow to the southwest in the Lower Shallow Aquifer, if it exists, is seasonal and is impeded by low-conductivity materials. The applicant also considered post-construction

simulations of the site in reaching this conclusion (ML102450252). Despite the appearance of a small mound in the Lower Shallow Aquifer beneath the proposed units in simulations, ground water flows to the east-southeast toward the site boundary.

- The applicant's analysis of thermal buoyancy concludes that the temperature delta (i.e., the temperature difference between the mixture of spilled radwaste and ambient ground water) could be 2.5°C (4.5 °F) (or 4.9°C [8.8 °F] in a sensitivity case [ML092710096]), and this delta temperature "would not likely cause buoyancy." A release from the radwaste building would occur within the backfilled excavation that is expected to exhibit a downward hydraulic gradient from the Upper toward the Lower Shallow Aquifer.

- The applicant evaluated conditions that could lead to chelating agents enhancing migration in aquifers and found that conditions at the STP site made it unlikely that the complexation of radionuclides by organic chelating agents would significantly influence ground water pathways.

The conceptual model of the ground water pathway is for ground water from postulated releases at STP Units 3 and 4 to move to the east-southeast in the Upper and Lower Shallow Aquifer to a conservative exposure point represented by a hypothetical water-supply well along this pathway and located on the eastern boundary of the site. A ground water pathway to the west-southwest from proposed Unit 4 to the western site boundary is also assumed to be intercepted by a water-supply well.

NRC Staff's Technical Evaluation.

The staff reviewed FSAR Subsection 2.4S.13.1.2 and the proposed revisions. In the FSAR the applicant adopted the east-southeast directed ground water pathway within the Upper and Lower Shallow Aquifer and a west-southwest directed ground water pathway within the Upper Shallow Aquifer, as the plausible pathways for an accidental radioactive liquid effluent release into ground water. The applicant considered but eliminated the following list of alternative groundwater pathways, exposure points and transport processes:

- exposure at or attributed to the relief wells
- exposure from a Deep Aquifer pathway
- exposure at the western boundary of the STP site from the Lower Shallow Aquifer
- a pathway related to thermal buoyancy
- enhanced transport because of the presence of chelating agents

The staff reviewed and accepts the east-southeast and west-southwest pathways described by the applicant as plausible groundwater pathways. Site characterization data and simulations are sufficient to support this conclusion. With regard to the alternative ground water pathways, exposure points, and transport processes eliminated by the applicant, the staff reviewed each alternative and concluded the following:

- The staff reviewed the applicant's supplemental information in the responses to RAIs. These RAIs discussed the relief wells (ML081960070) and the potentiometric surface in the vicinity of the main cooling reservoir. The staff concluded that ground water moves away from the main cooling reservoir and into the Upper Shallow Aquifer (ML092710096 and ML093310392), and the staff accepted the elimination of relief wells as an exposure point.

- The staff reviewed the potential for exposure via the Deep Aquifer and acknowledged the substantial separation between the Lower Shallow Aquifer and the Deep Aquifer, and the potentiometric data demonstrating that flow within the Deep Aquifer beneath the STP site is toward the STP production wells. The applicant acknowledges that the increase in ground water production consistent with the construction and operation of STP Units 3 and 4 will create lower potentiometric levels in the Deep Aquifer, a larger cone of depression, and an expanded area of lower potentiometric head over most of the northern portion of the STP site. The staff concluded that the vertical hydraulic gradient will increase, thereby causing a shorter travel time through the 30- to 46-m (100- to 150-ft) thick confining strata that separate the Shallow and Deep Aquifer. However, any release into the Deep Aquifer would be drawn into STP production wells. The staff also accepted the concept that releases into the Shallow Aquifer will likely travel in and discharge from the Shallow Aquifer to adjacent surface waters, rather than move into the Deep Aquifer.

- The staff reviewed the alternative pathway in the Lower Shallow Aquifer to the southwest of STP Units 3 and 4. The staff found that a southwest pathway and exposure point on the western site boundary is not plausible in the Lower Shallow Aquifer for the following reasons:

  - The evaluation of hydraulic properties in the region to the west-southwest of proposed STP Units 3 and 4 and the evaluation of the continuity of geohydrologic units in this region of the Lower Shallow Aquifer suggest that ground water movement from the proposed units will be less likely to occur to the west-southwest than to the east-southeast.

  - Potentiometric data for the Lower Shallow Aquifer in the vicinity of STP Units 1 and 2 show a flattening of the potentiometric surface and perhaps a very localized and low groundwater mound. The STP Units 3 and 4 excavations will create communication between the Upper and Lower Shallow Aquifers and will likely create a higher potentiometric surface in the Lower Shallow Aquifer at the postulated source release point. Simulations of the mound underlying Units 3 and 4 do not suggest that a west-southwest pathway will develop. Based on current information, the staff acknowledged that a Lower Shallow Aquifer pathway will likely move beneath the main cooling reservoir before crossing the site boundary to the east.

  - The ground water model and pathway analyses upon which the applicant based the plausible pathway decision were revised and provided to the staff. Supplemental RAI responses (ML103540324) were provided to the NRC on December 15, 2010. The revised groundwater model document (ML110140173) was provided to the NRC on January 11, 2011. The results of an alternative conceptual model using a spatially varying hydraulic conductivity distribution and the results of several sensitivity cases support elimination of a west-southwest pathway in the Lower Shallow Aquifer from the power block to the site boundary or LRS. These RAI responses and model documentation were reviewed by the staff and resulted in closing **Open Item 2.4.12-1**. (see Section 2.4S.12 of this SER).

- The staff reviewed the applicant's analysis of thermal buoyancy. The staff noted that the inclusion in the analysis of release and transport in the Upper Shallow Aquifer to the east-southeast and west-southwest made a further analysis of thermal buoyancy unnecessary, because the buoyancy-related pathway in the Upper Shallow Aquifer was included by the applicant.

- The staff reviewed the applicant's responses to RAI 02.04.12-17 (ML082270381) and RAI 02.04.13-7 (ML081970231). The staff accepted the applicant's conclusion that based on the unlikely release of chelating agents, substantial dilution by ground water, and the abundant source of competing cation clay and silt minerals, there will be a minimal potential for the enhancement of radionuclide migration due to the presence of chelating agents.

The staff's review of the applicant's information and data supporting the conceptual model topic confirmed that the applicant has addressed relevant information. The east-southeast and west-southwest directed pathways in the Upper Shallow Aquifer and the east-southeast directed pathway in the Lower Shallow Aquifer are accepted as plausible pathways with multiple exposure points. The applicant provided additional information relevant to the west-southwest pathway on December 15, 2010 (ML103540324), in response to RAIs issued in April 2010. The staff's reviews of these supplemental RAI responses and the revised ground water model documentation (ML110140173) resulted in closing **Open Item 2.4.12-1.** The staff concluded that the west-southwest directed pathway in the Lower Shallow Aquifer can be excluded.

2.4S.13.4.4 Analysis of Accidental Releases to Groundwater

Information Submitted by Applicant

In FSAR Subsection 2.4S.13.1.3, and its proposed revisions, the applicant describes the application of an approach to estimating radioactive contaminant concentrations in the groundwater pathway resulting from the postulated release from an LCW collection tank into groundwater surrounding the radwaste building. The applicant presents a two-step, one-dimensional calculation. In this calculation, the applicant considers parent and progeny radionuclides that are expected to be present in the LCW tanks and groundwater pathways. First, a calculation using the groundwater travel time (i.e., unretarded travel time) and decay is performed as a simple screen to eliminate radioisotopes that will have little effect on the public because they have short half-lives relative to the groundwater travel time. Second, a calculation using the transport processes of advection, dispersion, retardation, and decay is performed to provide a more realistic, yet still conservative, analysis of radioisotope concentrations at exposure points.

The applicant provides progeny radioisotopes in FSAR Figure 2.4S.13-1, and includes members of each decay chain identified by International Commission on Radiation Protection Publication 38 (ICRP 1983) to be considered in dose calculations. The results of the two-step calculation process are compared to the maximum permissible concentrations (i.e., the effluent concentration limits or ECLs) found in 10 CFR Part 20, Appendix B, Table 2, Column 2. The applicant applied progressively more realistic and less conservative assumptions to show compliance in the second step, considering only those radionuclides for which the results of the first step produced radioisotope concentrations greater than or equal to 1 percent of the ECL.

The first step is a screening calculation to identify radioisotopes to be further analyzed, and it assumes all the radionuclides migrate at the same rate as the groundwater. This assumption allows the Bateman equations as given in FSAR Equation 2.4S.13-8, -9, and -10, and in Appendix B of NUREG/CR–5512, Vol. 1 (Kennedy and Strenge, 1992) to be applied to the parent and first and second progeny.

The second step uses a standard equation and solution for one-dimensional transport along a groundwater pathline that includes the processes of advection, dispersion, retardation, and radioactive decay. The analytical solution is taken from Water Resources Monograph 10 published by the American Geophysical Union (Javandel et al., 1984).

The applicant performed the first step screening calculation on the groundwater pathway directed to the east-southeast of STP Unit 3 and on the exposure point at the eastern site boundary. Because all other east-southeast exposure points are on the same pathway but are farther from the source, the results of an analysis of the eastern site boundary exposure point are conservative for all exposure points considered. The applicant used both the representative average linear groundwater velocity reported in FSAR Section 2.4S.12 (see FSAR Tables 2.4S.12-14 and 2.4S.12-17), and the high estimate of linear groundwater velocity in the calculations. For the east-southeasterly pathway from Unit 3, the results of the screening analysis using the representative linear groundwater velocity identified radionuclides Ni-63, Sr-90, Y-90, Cs-137, and Pu-239 as analytes for further analysis. An analysis using the higher linear groundwater velocity and the lower travel time identified these radionuclides plus H-3 and Co-60 as analytes for further analysis. The analysis of the west-southwesterly pathway from Unit 4 for the representative linear groundwater velocity identified radionuclides Ni-63, Sr-90, Cs-137, and Pu-239 as analytes for further analysis. The use of the higher linear groundwater velocity identified H-3 and Y-90 as additional radionuclides for further analysis.

The second calculation step yields a more realistic and less conservative estimate of radionuclide concentration. Distribution coefficients for Co, Ni, Sr, Cs, and Pu were taken from a site-specific study, and the geometric mean of the lognormal distribution was used in the analysis as a "best" representation the geochemistry of Shallow Aquifer sediments. For the analyte tritium (H-3), there is no adsorption and its distribution coefficient was assigned a zero value. For the analyte yttrium (Y-90), there are no site-specific measurements. Its adsorption was assumed to be similar to that of scandium, an element adjacent to yttrium in the periodic table and estimated from literature values for scandium. For the purpose of conservatism, distribution coefficient values taken from the literature used the lowest 10th percentile probability value in the analysis. For all analytes analyzed in the Upper Shallow Aquifer pathway the dispersivity, total porosity, effective porosity, and bulk density values used in the analysis were 15.3 m (50.3 ft), 0.41, 0.33, and 1.58 g/cc (98.6 lb/ft^3), respectively. For all analytes analyzed in the Lower Shallow Aquifer pathway, these values were 15.3 m (50.3 ft), 0.39, 0.31, and 1.63 g/cc (101.8 lb/ft^3), respectively. The second calculation step for representative estimates of linear groundwater velocity and for both east and west directed pathways found no effluent concentration limit (ECL) violations and no sum of fraction violations at the eastern and western site boundary.

NRC Staff's Technical Evaluation

The staff reviewed FSAR Subsection 2.4S.13.1.3, and its proposed revisions. The staff confirmed that the applicant has addressed relevant information. The pathway analysis of an accidental release into groundwater for the STP site described by the applicant in FSAR Subsection 2.4S.13.1.3, focuses on three pathways: an east-southeast directed Upper Shallow Aquifer pathway, an east-southeast directed Lower Shallow Aquifer pathway, and a west-southwest directed Upper Shallow Aquifer pathway. Each is conservatively represented by exposure via a water-supply well at the respective site boundary, east and west. The staff reviewed the two-step analysis methodology presented by the applicant and reviewed its application to the three aquifer pathways.

The staff noted that the full decay chains do not appear to have been analyzed in the first step of the analysis. For example, the results of the analysis in revised FSAR Tables 2.4S.13-2A and 2.4S.13-2B (ML093310392) do~~es~~ not include the long-lived isotopes resulting from the Mo-99 and Te-129m decay chains, which include Tc-99 and I-129 shown in FSAR Figure 2.4S.13. The applicant's response to RAI 02.04.13-8 (ML082270381) states that the decay chains were truncated at a "progeny member where incremental dose from the total energy from all radiation emitted over a 100-year period is not significant." The staff independently confirmed the applicant's truncation process and found that the complete conversion of Mo-99 to Tc-99 and of Te-129m to I-129 yields a negligible dose. Therefore, RAI 02.04.13-8 is considered closed.

The staff reviewed the first and second step of the ground water pathway analysis and accepts the methodology and the hydraulic and geochemical data applied. The staff performed an independent calculation for both steps using the Bateman equations (Kennedy and Strenge 1992) and a more conservative approach omitting the dispersion phenomena in the second step. Based on a representative analysis of the pathway with the shortest travel-time-the east-southeast pathway in the Upper Shallow Aquifer from Unit 3 to a hypothetical water-supply well on the eastern boundary of the site—the first step of the screening analysis correctly identified analytes for further analysis. The second step, which included adsorption phenomena, showed no ECL violations and no sum of fractions violations.

The analysis of the radioactive liquid effluent transport through the groundwater pathway relies on all plausible pathways being identified for analysis. Supplemental responses to RAI 02.04.12-46 on the spatial bias in the model results, RAI 02.04.12-48 Part 1 on post-construction infiltration, and RAI 02.04.12-50 on groundwater mounding in the Lower Shallow Aquifer, were received on December 15, 2010 (ML103540324) and the revised ground water model document was received on January 11, 2011 (ML110140173). The staff's review of these submittals fully supports the pathways previously identified by the applicant, supports the applicant's initial position of dismissing a west-southwest directed pathway in the Lower Shallow Aquifer, and supports closure of **Open Item 2.4.12-1**.

2.4S.13.4.5 Compliance with 10 CFR Part 20

Information Submitted by Applicant

In FSAR Subsection 2.4S.13.1.4, and its proposed revisions, the applicant describes the comparison of the analysis results to the requirements of 10 CFR Part 20. The applicant's analysis evaluated the postulated accidental release of radioactive liquid effluents from the LCW collection tanks in the radwaste building into the Upper and Lower Shallow Aquifer pathway to the east-southeast of STP Unit 3, and the Upper Shallow Aquifer pathway to the west-southwest of STP Unit 4. This analysis shows that each of the radioactive analytes is below its respective ECL at the plausible and conservative exposure point (i.e., the hypothetical water-supply well at the eastern or western site boundary). The applicant has taken the sum of fractions approach and using the estimated radionuclide concentrations, has shown that the sum of fractions is below 1 for each pathway.

The applicant also performs a sensitivity analysis using the range of average linear velocity (see FSAR Table 2.4S.12-17; note that this table was revised in ML093310392) and the range of distribution coefficients (FSAR Table 2.4S.13-3). The applicant pairs relatively extreme conditions of maximum groundwater velocity and minimum distribution coefficients (rapid migration) and minimum groundwater velocity and the maximum distribution coefficient (slow migration). Where site-specific distribution coefficients were not available, the applicant applies the upper and lower bounds of the 95 percent confidence interval from literature values (ML092610376). Results of the sensitivity analysis showed that no exceedance of ECLs occurs for the case of rapid migration, the limiting case. The applicant notes that the variability of the geologic depositional environment underlying the STP site—and the resulting discontinuous fine-grained mixtures of sediment—suggest that average and geometric mean values of properties best represent the STP site.

The applicant considers the analysis conservative for the following reasons:

- The analysis omits the processes of dilution during release and diffusion during transport, and both would cause concentrations to be reduced.

- The analysis assumes that no mitigative measures are taken to remove the radioactive source or to reduce radioactive concentrations in the ground water.

- Credit is not taken for design elements of the radwaste building and the overall radwaste system that should prevent the release from occurring.

- Because the radwaste building foundation is below the water table, the release from a leaking exterior wall would require the building to first fill with ground water to the water table height. Until that time, ground water flow would be inward and the release could not occur. The time required would provide an opportunity for mitigative measures.

The applicant concludes that the STP site ground water pathway yielded an analysis that demonstrated compliance with 10 CFR Part 20, Appendix B, Table 2. Compliance was demonstrated for both individual radioisotopes and through the sum of fractions, for mixtures of radioisotopes.

NRC Staff's Technical Evaluation

The staff reviewed FSAR Subsection 2.4S.13.1.4 and its proposed revisions. The staff confirmed that the applicant has addressed relevant information. The staff reviewed the representative and sensitivity cases presented by the applicant for the accidental release of radioactive liquid effluent into the ground water. The representative case incorporates a one-dimensional model, and most properties are representative (i.e., the geometric mean of saturated hydraulic conductivity and the arithmetic mean of porosity). The hydraulic gradient is estimated as the high end of the observed range from the preconstruction piezometric surfaces. The sensitivity analysis performed by the applicant used the high end of the range of saturated hydraulic conductivities and the low end of the range of distribution coefficients to simulate the minimum travel-time case. This case also used the high estimate of the hydraulic gradient based on preconstruction data.

Simulations using the site groundwater model included the post-construction case of the confining layer between the Upper and Lower Shallow Aquifer being excavated and replaced with engineered backfill. Post-construction settings also examined the influence of a higher main cooling reservoir elevation. The ground water model simulations estimated travel times to the eastern site boundary within the range predicted by the one-dimensional model.

The staff performed an independent calculation to review ground water concentrations and the sum of fractions calculated by the applicant. The staff concurs that the results of the representative case and the minimum travel-time sensitivity case presented by the applicant comply with 10 CFR 20. The staff's review of the applicant's responses to RAIs associated with **Open Item 2.4.12-1** resulted in closing the open item. There were no revisions to FSAR Section 2.4S.13.

2.4S.13.4.6 Direct Releases to Surface Waters

Information Submitted by Applicant

In FSAR Subsection 2.4S.13.2, the applicant describes the credibility of flood events to result in a surface-water release from the radwaste building. The applicant notes that all tanks containing radioactive liquid effluents for STP Units 3 and 4 are inside the radwaste building, and there are no outdoor tanks in the liquid waste management system (LWMS). Notwithstanding the numerous design features of the radwaste building and radwaste system that make a release unlikely, the applicant determined that the most plausible accident scenario that could result in a release into surface water is a rapid and catastrophic flood such as a breach of the main cooling reservoir embankment (i.e., the design-basis flood), coinciding with leakage from the indoor tanks on the basement level of the radwaste building, (i.e., not unlike that described in FSAR Subsection 2.4S.13.1.1). Both of these events, (i.e., the design-basis flood and tank leakage within the radwaste building) are unlikely extreme events.

The applicant considers other external flood events to be slow-moving events that would allow ample warning and time to initiate actions that would mitigate the potential effects from flooding. Therefore, the applicant determined that none of the other external flood events was credible for use in the scenario of a direct release into surface water.

The applicant summarizes the effect of a coupled main cooling reservoir breach flood and radwaste building release event as follows:

- This magnitude of flooding would disperse and dilute the radionuclide concentration.
- There are no known users of the Colorado River or the LRS water downstream of the STP site.
- Therefore, no surface-water users would be affected.

In response to RAI 02.04.13-13 dated September 16, 2009 (ML092610376), the applicant uses main cooling reservoir breach flood and radwaste building release volumes and LCW radioisotope concentrations to quantify the level of radioactive contamination from the direct release of an accidental radioactive liquid effluent into surface waters. Using the 10 CFR Part 20 ECLs, the applicant demonstrated that the result of a main cooling reservoir breach flood and a coincident release from the radwaste building is a small fraction of the 10 CFR Part 20 limits.

NRC Staff's Technical Evaluation

The staff reviewed FSAR Subsection 2.4S.13.2, and its proposed revisions (ML 092610376). The applicant has stated there are no outdoor tanks in the LWMS; therefore, any accidental release of radioactive liquid effluent would come from a tank within a building inside the power block. The postulated release from the LCW collector tank in the radwaste building is such a release (see Subsection 2.4S.13.4.2 of this SER) and it represents an unlikely extreme event. All events resulting in surface water inundating all or a portion of the STP site are also unlikely extreme events, (e.g., storm surge, tsunami, dam breach, river flooding). Therefore, any direct release to surface water from an accidental release of radioactive liquid effluent would result from the combination of two unlikely extreme events. The staff determined that unlikely extreme events should not be combined. Therefore, there is no scenario for a direct release into surface water.

The postulated release to ground water, which was discussed in the preceding subsections of this section of the SER, would continue to move past the site boundary and eventually release to surface water, (e.g., the Colorado River). This represents an indirect release to surface water. However, such a release would experience additional environmental delay and dispersal, and, in the case of adsorbed contaminants, additional retardation and decay of the liquid effluent before being released to surface water. Accordingly, such an indirect accidental release of radioactive liquid effluent to surface water would involve lower concentrations than previously discussed and found acceptable.

In summary, the applicant has included sufficient relevant information to enable the staff's review of a direct release to surface water. The staff reviewed FSAR Section 2.4S.13, its proposed revisions, and the RAI responses. The staff concluded that there is no scenario for a direct release to surface water, and this portion of **Open Item 2.4.13-1** is closed.

2.4S.13.5 Post Combined License Activities

There are no post-COL activities related to this subsection.

2.4S.13.6 Conclusion

The staff reviewed the application and confirmed that the applicant has addressed the relevant information related to the effect of accidental releases of radioactive liquid effluent in ground and surface waters. As set forth above, the applicant has presented and substantiated information relative to the accidental releases of radioactive liquid effluent in ground and surface waters important to the design and siting of the proposed nuclear power plant.

The staff reviewed the available information. For the reasons given above, the staff concluded that the identification and consideration of the potential effects of accidental releases of radioactive liquid effluents in ground and surface waters on existing users and known and likely future users of ground and surface water resources in the vicinity of the site are acceptable and meet the requirements of 10 CFR 52.79, 10 CFR 100.23(d)(3), and 10 CFR 100.20(c) with respect to determining the acceptability of the site.

2.4S.13.7 References

10 CFR Part 20, Appendix B. *Code of Federal Regulations*. Title 10, Energy, Part 20 "Standards for Protection Against Radiation; Appendix B to Part 20, Annual Limits on Intake (ALIs) and Derived Air Concentrations (DACs) of Radionuclides for Occupational Exposure; Effluent Concentrations; Concentrations for Release to Sewerage"

International Commission on Radiation Protection (ICRP), 1983. "Radionuclide Transformations – Energy and Intensity Emissions, ICRP Publication 38, -13"

Javandel, I, C. Doughty, and C-F Tsang, 1984. Groundwater Transport: Handbook of Mathematical Models. Water Resources Monograph Series #10, American Geophysical Union, Washington, D.C.

Kennedy, W.E. and D.L. Strenge, 1992. Residual Radioactive Contamination From Decommissioning. NUREG/CR-5512, Vol. 1, U.S. Nuclear Regulatory Commission, Washington, D.C.

U.S. Nuclear Regulatory Commission, 2007. "Postulated Radioactive Releases Due to Liquid-Containing Tank Failures," Branch Technical Position 11-6, NUREG–0800, U.S. Nuclear Regulatory Commission, March 2007, Washington, D.C.

2.4S.14 Technical Specifications and Emergency Operation Requirements

2.4S.14.1 Introduction

This section of the FSAR describes the technical specifications and emergency operation requirements as necessary. The requirements described implement protection against floods for safety-related facilities to ensure that an adequate supply of water for shutdown and cool-down purposes is available.

This SER section provides an evaluation of the following specific areas: (1) controlling hydrological events, as determined in previous hydrology sections of the FSAR, to identify bases for emergency actions required during these events; (2) the amount of time available to initiate and complete emergency procedures before the onset of conditions while controlling hydrological events that may prevent such action; (3) reviewing technical specifications related to all emergency procedures required to ensure adequate plant safety from controlling hydrological events by the organization responsible for the review of issues related to technical specifications; (4) potential effects of seismic and non-seismic information on the postulated technical specifications and emergency operations for the proposed plant site; and (5) any additional information requirements prescribed in the "Contents of Application" sections of the applicable subparts of 10 CFR Part 52.

2.4S.14.2 Summary of Application

In Section 2.4S.14 the applicant addresses technical specifications and emergency operation requirements. In this section, the applicant provides site-specific supplemental information to address COL License Information Item 2.22 identified in DCD Tier 2, Revision 4, Section 2.3.

The applicant addressed the information as follows:

COL License Information Item

- COL License Information Item 2.22 Technical Specifications and Emergency Operation Requirement

COL License Information Item 2.20 requires the COL applicants to provide site-specific information related to flood-protection measures for STP Units 3 and 4 safety-related facilities and provisions to ensure that an adequate water supply is available to shut down and cool the reactor. The applicant provides supplemental information to establish technical specifications (TSs) and emergency operating procedures (EOPs) to ensure these measures. The applicant commits (COM 2.4S-1) that appropriate EOPs will include applicable provisions for the main cooling reservoir that are similar to those provided for STP Units 1 and 2, before fuel loading.

2.4S.14.3 Regulatory Basis

The relevant requirements of the Commission regulations for consideration of TSs and emergency protective measures, and the associated acceptance criteria, are in Section 2.4.14 of NUREG-0800.

The applicable regulatory requirements are as follows:

- 10 CFR Part 100, as it relates to identifying and evaluating hydrological features of the site. The requirement to consider physical site characteristics in site evaluations is specified in 10 CFR 100.20(c).

- 10 CFR 100.23(d)(3), as it sets forth the criteria to determine the siting factors for plant design bases with respect to seismically induced floods and water waves at the site.

- 10 CFR 52.79(a)(1)(iii), as it relates to identifying hydrologic site characteristics with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

- 10 CFR 50.36, as it relates to identifying technical specifications related to all emergency procedures required to ensure adequate plant safety from controlling hydrological events by the organization responsible for the review of issues related to technical specifications.

In addition, the staff used the regulatory positions of the following regulatory guides for the identified acceptance criteria:

- RG 1.59, as supplemented by the current best practices, provides guidance for developing the hydrometeorological design bases.

- RG 1.102 describes acceptable flood protection to prevent the safety-related facilities from being adversely affected.

2.4S.14.4 Technical Evaluation

The staff reviewed the information in Section 2.4S.14 of the STP Units 3 and 4 COL FSAR. The staff's review confirmed that the information in the application addresses the relevant information related to the TSs and EOPs. The staff's technical review of this section included an independent review of the applicant's information in the FSAR and in the responses to the RAIs. The staff supplemented this information with other publicly available sources of data.

This section describes the staff's evaluation of the technical information presented by the applicant in FSAR Section 2.4S.14.

COL License Information Item

- COL License Information Item 2.22 — Technical Specifications and Emergency Operation Requirement

The staff reviewed the applicant's supplemental information on the TSs and EOPs. The staff's review of the application is summarized below:

Information Submitted by Applicant

The applicant states that safe plant operations for STP Units 3 and 4 will not be affected by floodwater elevations, because all required systems and equipment are protected against the design-basis flood and will therefore remain operational during such an event.

The applicant states that the design-basis flood elevation of 12.2 m (40 ft) MSL is the result of a postulated failure of the main cooling reservoir embankment. Site grades in the power block area of STP Units 3 and 4 range from 9.8 to 11.2 m (32 to 36.6 ft) MSL, and the top of the concrete floor elevation of the structures located within the power block area is at 10.7 m (35 ft) MSL.

The applicant states that the structural and watertight flood-protection measures are applied to any STP Units 3 and 4 facilities that have an open passageway to any safety-related facility. For all facilities, watertight doors and hatches that are located below 12.2 m (40 ft) MSL will remain closed and under administrative control. Therefore, the applicant concludes that no EOPs or plant TSs are required to implement flood protection for STP Units 3 and 4.

The applicant states that with the exception of the main cooling reservoir embankment breach, flooding at the STP site is not a sudden event. During precipitation-induced flooding, the rise in river water elevation is gradual and slow. The approach of a hurricane can be forecasted and its trajectory can be tracked. The applicant estimates the shortest warning time during a postulated upstream dam failure on the Colorado River as 58 hours in FSAR Section 2.4S.4. Consequently, the applicant concludes that adequate time is available to implement remedial or preventive measures for non-safety-related facilities.

The applicant states that no emergency protective measures are needed for low-water events. Other than a major breach of its embankment, a drop in water surface elevation in the main cooling reservoir will be gradual. The only safety-related water reservoirs proposed for STP Units 3 and 4 are the two engineered, partially buried UHS water-storage tanks (FSAR Figures 2.5S.4-49A through 2.5S.4-49D). The two UHS water-storage tanks, one for each proposed unit, will be located south of the respective units. The capacity of these UHS water-storage tanks will be sufficient to meet 30 days of cooling requirements under design-basis accident conditions, without needing any makeup or blowdown.

NRC Staff's Technical Evaluation

The staff issued RAI 02.04.14-1 requesting the applicant to describe severe hydrology-related events (levee breach, heavy rain, hurricane, tsunami, etc.) and to provide a summary of maximum water levels and available lead times to initiate and complete emergency procedures for each event in preparation for the main cooling reservoir EOPs in the future.

The applicant responded to RAI 02.04.14-1 in a letter dated January 28, 2009 (ML090300648). The applicant's response provides a list of events with the associated maximum water surface elevations and corresponding lead times at the STP Units 3 and 4 site. Table 2.4S.14-1 below summarizes this information.

Table 2.4S.14-1 Hydrological Events that Produce High Water Surface Elevations at STP Units 3 and 4 Site and Corresponding Lead Times

Hydrological Event	Water Surface	Lead Time for Action	Basis for Determination of

	Elevation (m / ft MSL)		Lead Time
Postulated main cooling reservoir embankment breach	12.2 / 40	Greater than 30 minutes	Observation of main cooling reservoir conditions
Local intense precipitation	11.2 / 36.6	Greater than 2 hours	Flash flood or storm warnings from the National Weather Service
Multiple concurrent upstream dam failures	10.5 / 34.4	Between 58 and 65 hours	Notification from the Lower Colorado River Authority
Probable maximum flood in the Colorado River Basin	8.0 / 26.3	Flood does not reach site grade	Notification from the Lower Colorado River Authority
Probable maximum tsunami	3.5 / 11.5	Flood does not reach site grade	Post-event notification
Probable maximum hurricane	9.5 / 31.1	Flood does not reach site grade	Real-time monitoring by the National Hurricane Center

The applicant states that with the exception of the flood resulting from the main cooling reservoir embankment failure, sufficient time will be available to carry out site preparation activities such as ensuring an adequate supply of fuel oil, reducing floor drain sump inventories, ensuring the availability of sufficient maintenance personnel, ensuring the operation of emergency communication systems, sandbagging non-watertight entrances to buildings that are not safety-related, restoring watertight seals, and reducing low-level liquid waste inventories.

The applicant further states that emergency procedures for the main cooling reservoir breach will require closing watertight doors that are normally open and providing access to the control building. The applicant states that this is typically the only action necessary to ensure that safety-related equipment is safe from severe hydrology-related events.

The staff reviewed the applicant's information and determined that it is sufficient for future preparations of EOPs related to severe hydrology events. Therefore, RAI 02.04.14-1 is considered closed.

2.4S.14.5 Post Combined License Activities

The applicant identifies the following commitment:

- Commitment (COM 2.4S-1) – Develop EOPs for the main cooling reservoir that are similar to those provided for STP Units 1 and 2, before fuel loading.

2.4S.14.6 Conclusion

The staff reviewed the application and confirmed that the applicant has addressed the information relevant to technical specification and emergency operations requirements. Based on the applicant's information, the staff determined that the main cooling reservoir embankment breach is the only severe hydrology-related event that may require EOPs. Therefore, no outstanding information is required to be addressed in the COL FSAR related to this section.

As set forth above, the applicant has presented and substantiated information relative to the TSs and EOPs important to the operation of this plant. The staff accepted the methodologies used to determine the TSs and emergency operations, as documented in SERs for previous licensing actions. Therefore, the staff found that the information addressing COL License Information Item 2.22 is adequate and acceptable. The staff concluded that the identified TSs and emergency operations meet the requirements of 10 CFR 50.36, 10 CFR 52.79, 10 CFR 100.23(d), and 10 CFR 100.20(c).

2.4S.14.7 References

South Texas Nuclear Operating Company, "South Texas Project Combined License Application," Revision 0, Part 2, Final Safety Analysis Report, 2007.