Luminant

Rafael Flores
Senior Vice President &
Chief Nuclear Officer
rafael.flores@luminant.com

Luminant Power
P O Box 1002
6322 North FM 56
Glen Rose, TX 76043

T 254.897.5590
F 254.897.6652
C 817.559.0403

CP-201101181
Log # TXNB-11057

Ref. # 10 CFR 52

August 29, 2011

U. S. Nuclear Regulatory Commission
Document Control Desk
Washington, DC 20555
ATTN: David B. Matthews, Director
Division of New Reactor Licensing

SUBJECT: COMANCHE PEAK NUCLEAR POWER PLANT, UNITS 3 AND 4
DOCKET NUMBERS 52-034 AND 52-035
RESPONSE TO HYDROLOGY SAFETY REVIEW OPEN ITEMS
(SECTIONS 2.4.12, 2.4.13, AND 2.5.4)

Dear Sir:

The NRC conducted an audit for the Comanche Peak Nuclear Power Plant Units 3 and 4 Combined License Application hydrology safety review on June 7-9, 2011. The audit was to resolve a number of open items resulting from Luminant's responses to preliminary requests for additional information (RAIs) issued as part of the acceptance review and RAIs issued as part of Phase 1 of the safety review. Luminant has addressed the audit open items in this letter by providing supplemental responses to six NRC questions from three RAIs.

Attachment 1 includes the six questions and their supplemental responses, and Attachment 2 provides the marked-up pages that incorporate the supplemental responses into the Final Safety Analysis Report (FSAR). For each mark-up, the associated supplemental response is annotated in the right margin. To assist the reviewer, the context of the marked-up pages is enhanced by including adjacent pages which may not have been changed.

There are no commitments in this letter. Should you have any questions regarding this response, please contact Don Woodlan (254-897-6887, Donald.Woodlan@luminant.com) or me.

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

Executed on August 29, 2011.

Sincerely,

Luminant Generation Company LLC

Donald R. Woodlan for

Rafael Flores

Attachments: 1. Supplemental Responses to Requests for Additional Information
2. FSAR Pages Incorporating the Supplemental Responses

D090
NRO

Electronic distribution w/attachments:

Rafael.Flores@luminant.com
mlucas3@luminant.com
jeff.simmons@energyfutureholdings.com
Bill.Moore@luminant.com
Brock.Degeyter@energyfutureholdings.com
rbird1@luminant.com
Allan.Koenig@luminant.com
Timothy.Clouser@luminant.com
Ronald.Carver@luminant.com
David.Volkening@luminant.com
Bruce.Turner@luminant.com
Eric.Evans@luminant.com
Robert.Reible@luminant.com
donald.woodlan@luminant.com
John.Conly@luminant.com
JCaldwell@luminant.com
David.Beshear@txu.com
Ashley.Monts@luminant.com
Fred.Madden@luminant.com
Dennis.Buschbaum@luminant.com
Carolyn.Cosentino@luminant.com
NuBuild Licensing files
sfrantz@morganlewis.com
jrund@morganlewis.com
tmatthews@morganlewis.com
regina.borsh@dom.com
diane.aitken@dom.com
askolhek@bechtel.com
yoshinori_fujiwara@mhi.co.jp
kano_saito@mhi.co.jp
shigemitsu_suzuki@mhi.co.jp
Luminant Records Management (.pdf files only)
shinji_kawanago@mnes-us.com
masanori_onozuka@mnes-us.com
ck_paulson@mnes-us.com
joseph_tapia@mnes-us.com
russell_bywater@mnes-us.com
william_mcconaghy@mnes-us.com
mutsumi_ishida@mnes-us.com
yukako_hill@mnes-us.com
nicholas_kellenberger@mnes-us.com
ryan_sprengel@mnes-us.com
al_freitag@mnes-us.com
masaya_hoshi@mnes-us.com
rjb@nei.org
kak@nei.org
michael.takacs@nrc.gov
cp34update@certrec.com
michael.johnson@nrc.gov
David.Matthews@nrc.gov
Balwant.Singal@nrc.gov
Hossein.Hamzehee@nrc.gov
Stephen.Monarque@nrc.gov
jeff.ciocco@nrc.gov
michael.willingham@nrc.gov
john.kramer@nrc.gov
Brian.Tindell@nrc.gov
Alicia.Williamson@nrc.gov
Elmo.Collins@nrc.gov
Loren.Plisco@nrc.com
Susan.Vrahoretis@nrc.gov
Frank.Akstulewicz@nrc.gov
ComanchePeakCOL.Resource@nrc.gov

Attachment 1

Supplemental Responses to Requests for Additional Information

RAI 4314 (CP RAI #147) Question 02.04.12-8 S01
Question 02.04.12-9 S02

RAI 4315 (CP RAI #145) Question 02.04.13-5 S01
Question 02.04.13-6 S01
Question 02.04.13-7 S01

RAI 2929 (CP RAI #22) Question 02.05.04-11 S01

SUPPLEMENTAL RESPONSE TO REQUEST FOR ADDITIONAL INFORMATION

Comanche Peak, Units 3 and 4

Luminant Generation Company LLC

Docket Nos. 52-034 and 52-035

RAI NO.: 4314 (CP RAI #147)

SRP SECTION: 02.04.12 - Groundwater

QUESTIONS for Hydrologic Engineering Branch (RHEB)

DATE OF RAI ISSUE: 2/26/2010

QUESTION NO.: 02.04.12-8 S01

NUREG-0800, Standard Review Plan (SRP), Chapter 2.4.12, 'Groundwater,' establishes criteria that the NRC staff intends to use to evaluate whether an applicant meets the NRC's regulations.

By letter dated October 2, 2009, the NRC staff issued RAI ID 3672 (RAI No. 114) Question Number 14266 (02.04.12-1), in which the NRC staff asked, "Provide a description of the process followed to determine the conceptual models subsequently used to establish subsurface site characteristics related to groundwater to ensure that the most conservative of plausible conceptual models have been identified."

The applicant responded in document CP-200901564-Log No TXNB-09067-(ML093230704) executed on November 13, 2009. The NRC staff has reviewed the response and has determined that additional information is needed in order to complete its review.

The staff determined that an adequate description of the processes used to develop conservative conceptual models used subsequently in the accidental release evaluations was not sufficiently provided in the RAI response. The information provided in the response has numerous assumptions and lacked adequate conceptual description, data, and analyses to characterize the site alterations and how these alterations affect the hydrologic processes at the site. For example, it is assumed that there will not be any shallow groundwater at the site after construction is completed because the A and B zones will be entirely removed and the surface water drainage system will be designed to prevent subsurface infiltration.

Also, the NRC staff disagrees with a statement, which was made intermittently throughout the RAI responses and the combined license application, Revision 1 Part 2 FSAR, that groundwater within the Glen Rose Formation is "not real groundwater". This statement is unsupported since on the basis of data presented, the NRC staff asserts that the Glen Rose Formation is indeed a groundwater bearing perched aquifer.

In order to make its safety determination based on adequate characterization of the site, the NRC staff requests that the applicant provide the information below. The responses should follow guidance related to the analysis of groundwater related hazards through compliance with this and the accompanying RAIs.

1. Provide adequate conceptual and site specific information on how the surface water and groundwater flow system is expected to change after Comanche Peak Nuclear Power Plant, Units 3 and 4 are constructed.

2. Provide an adequate site conceptual model supported by data, analyses and construction design information to support the conclusions presented.

This is supplemental RAI 2.4.12-00-S.

SUPPLEMENTAL INFORMATION:

This supplemental information is provided to address NRC Hydrologic Open Item 2.4.12-1 from the audit conducted June 7-9, 2011. This supplemental information expands the original response to Question 2.4.12-8 as a result of discussions during the audit.

FSAR Subsections 2.4.12.2.4, 2.4.12.2.5.2, 2.4.12.3, and 2.4.12.5 were revised to provide a clearer picture of the post-construction site conceptual groundwater model and alternate groundwater pathways.

Impact on R-COLA

See attached marked-up FSAR Revision 2 pages 2-lix, 2-lx, 2.4-75, 2.4-77, 2.4-78, 2.4-79, 2.4-80, 2.4-81, 2.4-82, 2.4-83, 2.4-84, 2.4-85, 2.4-86, 2.4-87, 2.4-88, 2.4-89, 2.4-90, 2.4-91, and 2.4-241; Figures 2.4.12-212, 2.4.12-213, 2.4.12-214, and 2.4.13-201; and new Figures 2.4.12-215 and 2.4.12-216. Some of the mark-ups are editorial in nature and are annotated with a "CTS" number rather than the RAI question number.

Impact on S-COLA

None; this is a site-specific response.

Impact on DCD

None.

SUPPLEMENTAL RESPONSE TO REQUEST FOR ADDITIONAL INFORMATION

Comanche Peak, Units 3 and 4

Luminant Generation Company LLC

Docket Nos. 52-034 and 52-035

RAI NO.: 4314 (CP RAI #147)

SRP SECTION: 02.04.12 - Groundwater

QUESTIONS for Hydrologic Engineering Branch (RHEB)

DATE OF RAI ISSUE: 2/26/2010

QUESTION NO.: 02.04.12-9 S02

NUREG-0800, Standard Review Plan (SRP), Chapter 2.4.12, 'Groundwater,' establishes criteria that staff intends to use to evaluate whether an applicant meets the NRC's regulations.

By letter dated October 2, 2009, the NRC staff issued RAI ID 3672 (RAI No. 114) Question Number 14267 (02.04.12-2), in which the NRC staff asked "The CPNPP Units 1 and 2 FSAR states that alterations related to construction increased groundwater levels onsite. In order to understand the effect of construction of Units 3 and 4 on the hydrologic characteristics of the subsurface, plausible groundwater pathways, and site groundwater levels, Luminant is requested to provide a detailed description of the location and extent of planned construction activities including: excavation of regolith/undifferentiated fill and bedrock, the placement of engineered fill and the addition of engineered features (such as drainage ditches, parking lots, roads, etc.). Additionally, please evaluate and discuss the impact of these changes on site hydrologic processes such as infiltration, surface runoff, groundwater levels, hydraulic gradients and flow paths."

The applicant responded in document CP-200901564-Log No TXNB-09067-(ML093230704) executed on November 13, 2009. The NRC staff has reviewed the response and has determined that additional information is needed in order to complete its review.

The staff acknowledges that the additional information provided in the response partially satisfies the information need with regard to the post-construction site conditions. However, the information provided did not incorporate adequate description of the location and extent of planned construction activities including: excavation of regolith, undifferentiated fill and bedrock, the placement of engineered fill and the addition of engineered features (such as drainage ditches, subsurface drains, parking lots, roads, etc.)

The NRC staff provides the following examples that demonstrate some of the inadequacies in the description and level of details provided within the response.

1) The applicant stated that there will not be any shallow groundwater at the site after construction is completed because the A and B zones will be removed and the surface water drainage system will be designed to prevent subsurface infiltration and preclude buildup near plant foundations. However, these statements are not sufficient to illustrate that the system will function as designed and to establish a

maximum operational groundwater level and ensure compliance with the US-APWR design parameter groundwater level. In fact, Section 2.4.13 of the FSAR for Units 1 and 2 states that construction activities actually created areas where water levels were elevated due the placement of permeable fill materials. The data and evaluations presented are not adequate and of sufficient detail to show that this will not occur at the Units 3 and 4 site. For example, Figures 2.4.12-213 and 2.4.12-214 show new fill around many of the new structures but it is not clear how and if this new fill will be drained and what post-construction groundwater and surface water conditions (flow and levels) will be like.

2) The water level hydrographs from B-zone monitoring wells MW1201b (middle of Unit 4) and MW1207b (just north of Unit 3) have water level elevations of over 830 ft. The screened interval for these wells extends to elevations of 808 ft and 803 ft, respectively, which is well below the 822 ft site grade. This suggests that at least some portion of the water bearing B-zone could remain after the site grading is completed. The applicant has stated that it will all be removed.

In order to make its safety determination based on adequate characterization of the site that depicts the post-construction scenario adequately, the NRC staff requests that the applicant provide the following information.

1) A qualitative description of the construction related impacts that could affect site hydrology including maps at a legible scale, sufficiently detailed engineering design information on drainage systems and a description of conservative measurements or estimates of hydrologic parameters. This information should be of sufficient detail to support an analysis of the impact of site modifications on site hydrologic processes such as infiltration, surface runoff, groundwater levels, hydraulic gradients and flow paths.

2) A conservative quantitative analysis that demonstrates that the estimated maximum operational groundwater level complies with the US-APWR Design Certification Document.

This is supplemental RAI 2.4.12-01-S.

SUPPLEMENTAL INFORMATION:

This supplemental information is provided in response to NRC Hydrology Open Item 2.4.12-3 from the audit conducted June 7-9, 2011. It addresses NRC questions pertaining to potential horizontal groundwater migration along bedding planes that may not intersect the surface perimeter drain trench from current groundwater recharge locations (south and west) into the engineered fill within the plant area and the effect these pathways may have on post-construction groundwater elevations surrounding the subgrade safety-related structures.

The original response to Question No. 02.04.12-9 (ML102440679) stated the following:

> Once the soils (groundwater A-zone) are removed from the site, a topographically higher soil horizon will remain to the south and west of the power block areas. Although this higher soil horizon may provide a recharge source for those portions of the engineering layer "A" that remain surrounding the engineered fill within the plant site, the final site grading and drainage plan, illustrated in both FSAR Figures 2.4.2-202 and 2.5.5-204, shows the base of the drainage system along the southern and western boundaries of the plant site to be less than 820 ft msl. The northern and eastern boundaries of the site slope directly from the plant grade level of 822 ft msl towards Squaw Creek Reservoir (SCR), with normal pool elevation at 775 ft msl, or the stormwater holding ponds, with elevations at approximately 800 ft msl. This relationship is also depicted in Figure 1-1.

Groundwater recharge from the remaining soils into engineering layer "A" and the engineered fill surrounding the embedded portions of Units 3 and 4 is constrained by the base of the trench drain. With the exception of small amounts of landscaping, no soils would remain in the plant site which would be capable of recharging the remaining engineering layer "A". Site drainage is designed to remove stormwater with no pooling, thus preventing areas of recharge within the plant site.

Based on the interception of infiltrating perched groundwater or stormwater by the site drainage system, the maximum groundwater potentiometric surface at the site would be found at the base of the trench drain along the southern and western cut banks, equivalent to a maximum elevation of 820 ft msl. No constraining boundaries exist on the northern and eastern plant boundaries prior to the lower pool elevations of SCR. One retaining wall is to be constructed along the shore of SCR north of the Unit 3 UHS basins; however, the retaining wall is of limited extent and the top of the retaining wall is approximately 800 ft msl, well below the DCD maximum groundwater elevation. Two stormwater retention basins are also to be constructed to the east of Unit 3 and the west of Unit 4; however, the overflow elevations will be less than 810 ft msl, preventing accumulated stormwater from recharging the subsurface groundwater above the DCD maximum groundwater elevation.

Following the original response and the NRC Hydrology Audit, the site grading and drainage plan was modified, resulting in changes to the maximum invert elevation of the southern and western cut bank trench drains, retaining wall modifications, and the overflow elevations for the eastern stormwater retention basin as follows:

- Cut bank trench drain maximum invert elevation: 813.5 ft msl
- Additional retaining walls maximum elevation: 817 ft msl
- Eastern stormwater retention basin overflow elevation: less than 812 ft msl
- Western stormwater retention basin overflow elevation: less than 810 ft msl

Based on the changes to the cut bank trench drain elevation, the expected maximum groundwater potentiometric surface at the site would be found at the base of the trench drain along the southern and western cut banks, equivalent to a maximum elevation of 813.5 ft msl. Previous calculations (groundwater velocity, postulated accident analysis) used 820 ft msl for the maximum conservative groundwater elevation. Although this elevation has been superseded due to the grading and drainage plan changes, it provides a more conservative analysis (faster groundwater velocities and shorter travel times) than the new, lower elevation. Therefore, the previously published groundwater velocities and travel times to SCR will not be changed in the FSAR text as they are based on a more conservative analysis than the current site conditions dictate.

The following discussion expands the original response quoted above:

The extent of engineered fill placement is depicted on Figure 1-1. This figure shows the engineered fill within the Circulating Water System (CWS) piping trench[1] along the southern boundary of the plant site will underlie much of the southern cut bank. The CWS piping trench is shown between the western cut bank and the higher topographic areas to the west of the plant site. A significant portion of the CWS piping trench east of the Unit 3 turbine building is depicted as being in contact with the existing fill to the east of Unit 3 (see attached Figure 1-1). Due to its down-gradient position, the existing fill will provide a drainage location for the engineered fill.

---

[1] Descriptions of groundwater flow pathways in piping trenches included in this discussion, such as those pertaining to the CWS, describe groundwater within the engineered fill surrounding these subsurface structures.

Cross sections through the plant site (Figures 1-2 and 1-4) show the CWS piping trenches are embedded to the approximate depth of other sub grade structures and intersect any horizontal bedding planes present above the engineering "C" layer.

Horizontal seepage of groundwater along bedding planes from the south or west will intersect the CWS piping trenches prior to reaching the engineered fill surrounding the Auxiliary Building or Reactor Building and will be transported within the engineered fill to the existing fill. The engineered fill properties have not been designed, so the nature of the engineered/existing fill contact is unknown at this time.

The hydraulic conductivity of the eastern existing fill has been measured at 3.5×10^{-3} cm/sec or 74.2 gallons/day/ft^2 (gpd/ft^2). Therefore, for each square foot of contact between the existing and engineered fill, 74.2 gallons of groundwater can be drained from the engineered fill through the eastern existing fill to SCR.

The hydraulic conductivity of the western existing fill has been measured at 5×10^{-4} cm/sec or 10.6 gpd/ft^2. Therefore, for each square foot of contact between the existing and engineered fill, 10.6 gallons of groundwater can be drained from the engineered fill through the western existing fill to SCR.

Based on Figure 1-1, the easternmost contact wall between the CWS piping trench engineered fill and the eastern existing fill will be at least 350 ft wide (Figure 1-1) and 23.5 ft deep (maximum groundwater elevation at 813.5 ft msl minus elevation of the top of the piping trench fill concrete at 790 ft msl). Well MW-1214a (B2028), located near this contact location, reported the top of rock elevation at 781 ft msl. Therefore, this entire contact wall will be within the eastern engineered fill (Figure 1-6). This results in a contact surface area between the existing and engineered fill at that location of at least 8225 ft^2. This would be considered a minimum contact surface as excavation and stabilization requirements for installation of the CWS piping would most likely require additional excavation and placement of engineered fill or leveling concrete.

Based on the hydraulic conductivity of the eastern existing fill (74.2 gpd/ft^2), the drainage rate of the fully-saturated fill at the expected maximum groundwater elevation (813.5 ft msl) would be 610,295 gpd (424 gpm) through the existing/engineered fill contact at this location. Even at the minimum hydraulic conductivity (western existing fill 10.6 gpd/ft^2), this would allow a drainage rate of approximately 87,185 gpd (60.54 gpm).

Due to the unknown orientation, size, and properties of the contacts between the engineered fill and existing fill for Units 3 and 4, this assessment considers the drainage rate for only a small portion of the total engineered/existing fill contact surfaces present. The actual drainage rate is expected to be higher due to additional contact surfaces not considered in this assessment (Figure 1-1). Given the hydraulic conductivities of the existing fill areas, it is not expected that precipitation recharge into the southern or western soils or infiltration from stormwater ditches will exceed the potential drainage rate provided by the existing fill. However, for conservatism the groundwater travel time and velocity calculations for the existing fill assume the groundwater elevation will build up to a level of 820 ft msl, above the highest (most conservative) trench drain elevation (813.5 ft msl) at which elevation surface seepage will occur through the base of the trench drain, limiting the maximum water level with the engineered fill to 813.5 ft msl (approximately 814 ft msl). Brief mounding above 813.5 ft may occur due to short-term stormwater accumulation in the drainage trenches; however, the mounding would not exceed the tank failure analysis elevation (820 ft msl) or the DCD maximum allowable elevation (821 ft msl).

Based on this assessment, it is unlikely that preferential seepage from the western or southern soils would cause a rise in groundwater elevations within the engineered fill that would exceed the maximum allowed groundwater elevation of 821 ft bgs as defined by the DCD. Furthermore, groundwater flow through these additional drainage locations would result in longer groundwater travel pathways through the existing fill to SCR and would not affect the results of the conservative accident analysis presented in FSAR Subsection 2.4.13.

Attachments

Figure 1-1 Revised Cross-Section Location Map
Figure 1-2 Revised Cross-Section 3a
Figure 1-4 Revised Cross-Section 4a
Figure 1-6 Cross Section 3d SCR Elevation 770 ft msl.

Impact on R-COLA

None.

Impact on S-COLA

None; this response is site-specific.

Impact on DCD

None.

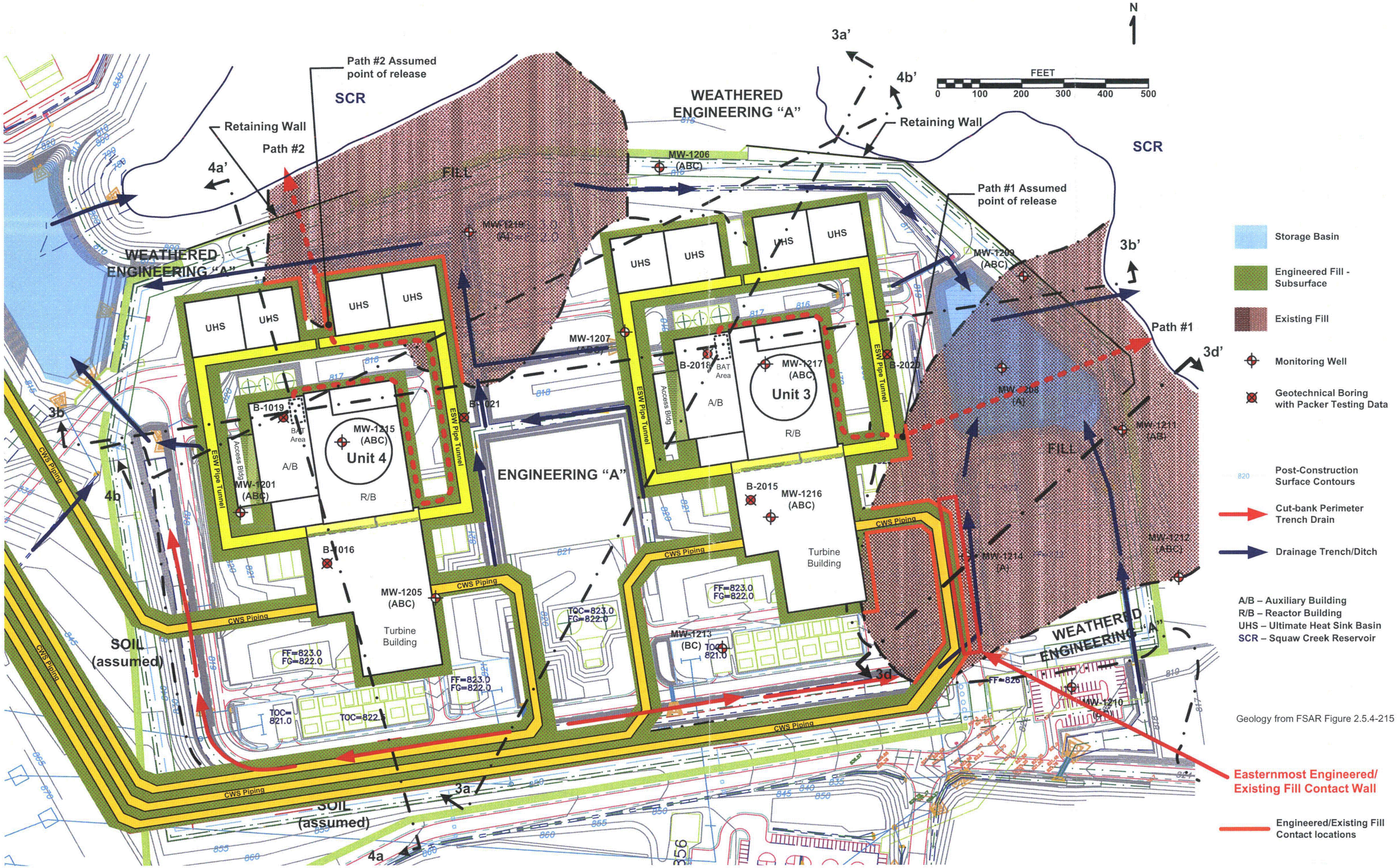


Figure 1-1 Revised Cross-Section Location Map

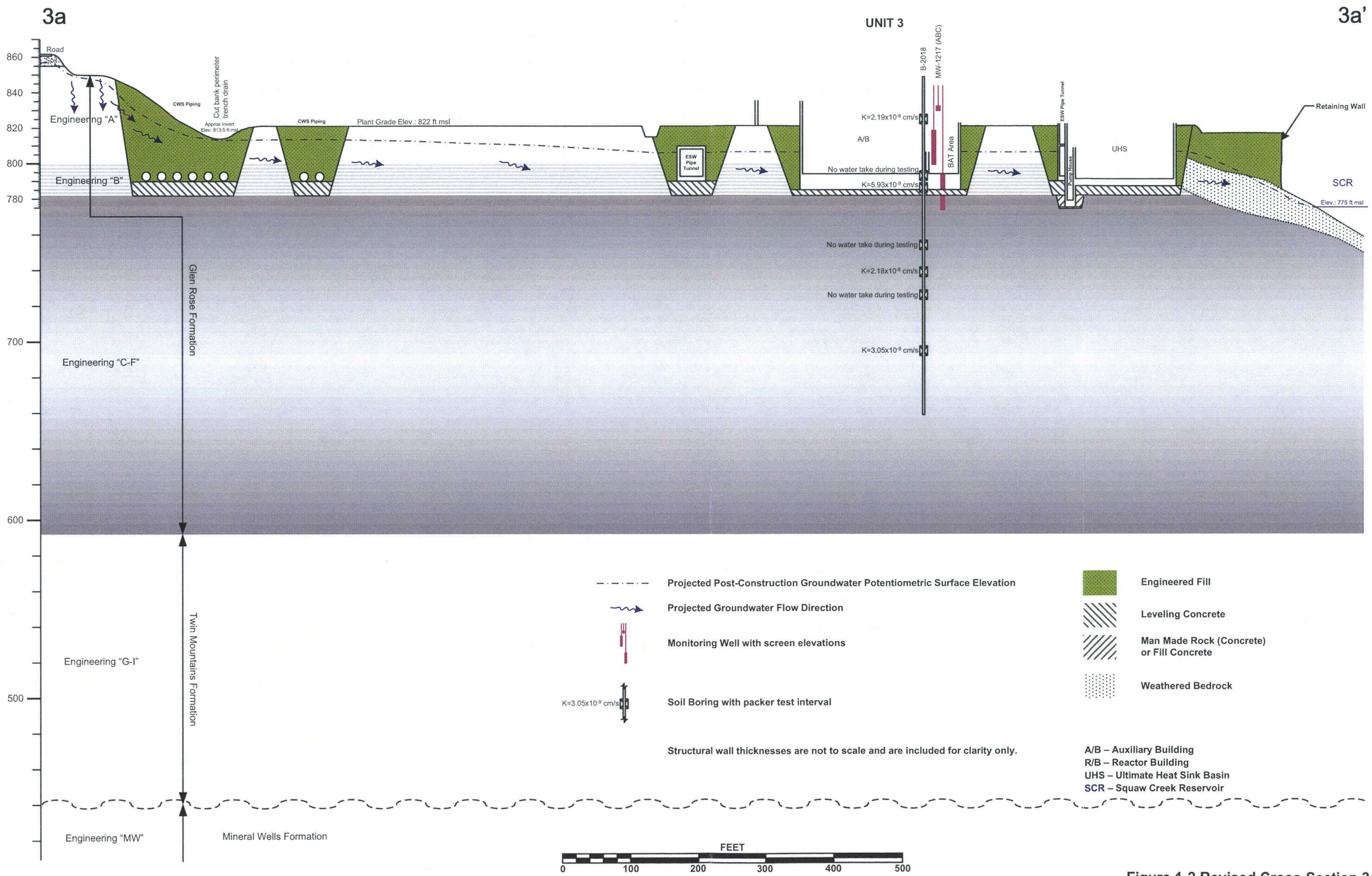


Figure 1-2 Revised Cross-Section 3a

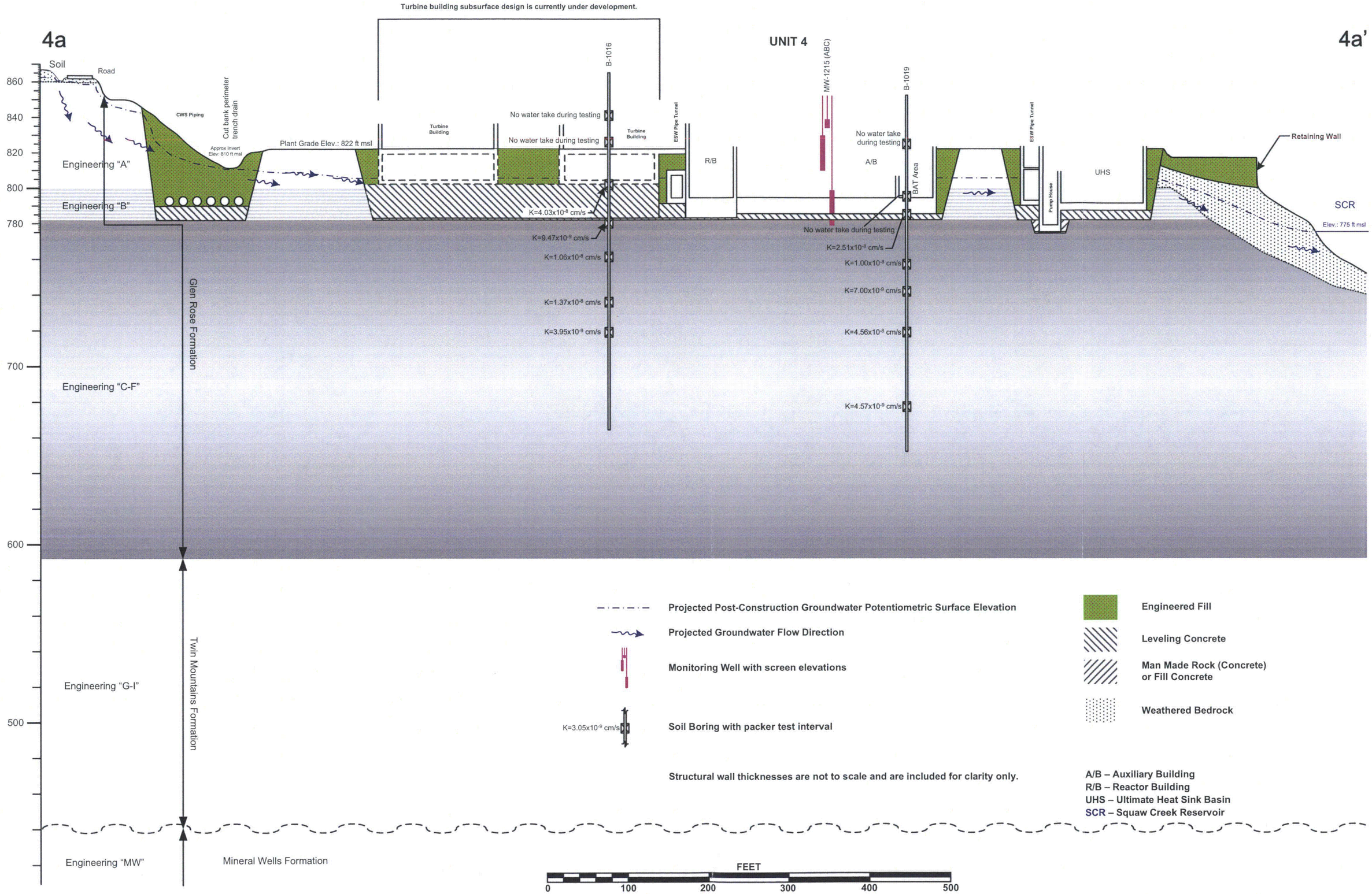
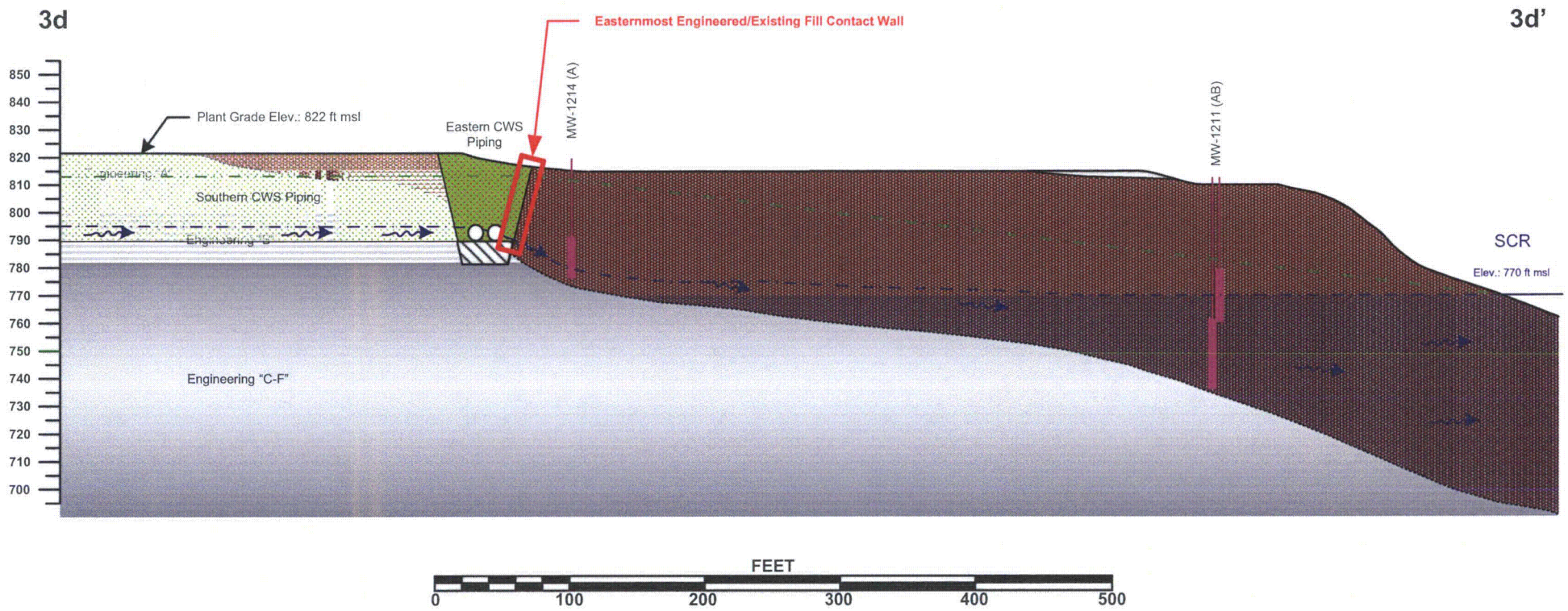


Figure 1-4 Revised Cross-Section 4a



Expected Post-Construction Groundwater Potentiometric Surface Elevation

Conservative Post-Construction Groundwater Potentiometric Surface Elevation

Projected Groundwater Flow Direction

Monitoring Well with screen elevations

Existing Fill

Weathered Bedrock

Existing Fill below the SCR pool elevation (770 ft. msl)

Figure 1-6
Cross Section 3d
SCR Elevation 770 ft msl

SUPPLEMENTAL RESPONSE TO REQUEST FOR ADDITIONAL INFORMATION

Comanche Peak, Units 3 and 4

Luminant Generation Company LLC

Docket Nos. 52-034 and 52-035

RAI NO.: 4315 (CP RAI #145)

SRP SECTION: 02.04.13 - Accidental Releases of Radioactive Liquid Effluents in Ground and Surface Waters

QUESTIONS for Hydrologic Engineering Branch (RHEB)

DATE OF RAI ISSUE: 2/26/2010

QUESTION NO.: 02.04.13-5 S01

NUREG-0800, Standard Review Plan (SRP), Chapter 2.4.13, 'Accidental Releases of Radioactive Liquid Effluents in Ground and Surface Waters,' establishes criteria that the NRC staff intends to use to evaluate whether an Applicant meets the NRC's regulations.

By letter dated October 5, 2009, the NRC staff issued RAI ID 3673 (RAI No. 116) Question Number 14273 (02.04.13-1), in which the NRC staff asked "Provide a description of the development of alternate conceptual models of the site and the process used in the selection of the most conservative and plausible pathway taking into consideration changes that will occur to site hydrology as a result of site alterations during construction."

The applicant responded in document CP-200901565-Log No TXNB-09068-(ML093230229) executed on November 16, 2009. The NRC staff has reviewed the response, as well as portions of Updated Tracking Report No. 4 referenced in the response, and has determined that additional information is needed in order to complete its review.

Similar to the applicant's response to RAI 3672 (RAI No. 114), this response does not adequately illustrate and discuss construction alterations, and the impact to the groundwater and surface water systems (e.g., groundwater levels and flowpaths). The NRC staff notes that SRP 2.4.13 states that alternative conceptual models should be developed and analyzed based on geologic and hydrologic characteristics of the site.

In order to make its safety determinations based on consideration of conservative parameters and scenarios for the transport of accidentally released radioactive liquid effluents, the NRC staff requests that the applicant provide conceptual models and selections for bounding sets of pathways that produce the most adverse contaminant concentrations to receptors in the analysis. Specifically the vertical migration pathway to the Twin Mountains Formation should be evaluated and calculations conducted to estimate potential concentrations at wells within the Twin Mountains Formation.

This is supplemental RAI 2.4.13-00-S.

QUESTION NO.: 02.04.13-6 S01

NUREG-0800, Standard Review Plan (SRP), Chapter 2.4.13, 'Accidental Releases of Radioactive Liquid Effluents in Ground and Surface Waters,' establishes criteria that the NRC staff intends to use to evaluate whether an Applicant meets the NRC's regulations.

By letter dated October 5, 2009, the NRC staff issued RAI ID 3673 (RAI No. 116) Question Number 14274 (02.04.13-2), in which the NRC staff asked "In order to demonstrate compliance with the requirements of providing adequate protection to water users, discuss the potential for preferential flowpaths and vertical migration and provide conservative evaluations and discussion of the potential for flow to offsite wells (displayed on Figure 2.4.-205). Also provide data and discuss the applicability of using the calculations performed as part of the FSAR for Units 1 and 2 as the basis to eliminate conceptual models of vertical groundwater flow through the Glen Rose to offsite wells in the Twin Mountains Formation from Units 3 and 4."

The applicant responded in document CP-200901565-Log No TXNB-09068-(ML093230229) executed on November 16, 2009. The NRC staff has reviewed the applicant's response and has determined that additional information is needed in order to complete its review.

The NRC staff notes that offsite groundwater wells located within the Twin Mountains Formation could be potential receptors of groundwater flowing from the site. The applicant's response to this RAI seeks to eliminate the vertical pathway to the Twin Mountains Formation based on analyses performed as part of the Comanche Peak Nuclear Power Plant, Unit 1 and 2 evaluations and included in the Units 1 and 2 FSAR. However, these calculations showed that flow to wells within the Twin Mountains Formation was possible within approximately 400 years and that the resultant concentration of 137-Cs was above the 10 CFR Part 20 Appendix B Effluent Concentration Limits (ECL), despite the 400 year travel time. In addition, vertical migration calculations conducted for Units 1 and 2 do not appear to incorporate conservative, site specific conditions encountered at Units 3 and 4. Through review of information published by the U.S. Geological Survey (USGS) and the Texas Water Development Board (TWDB), the NRC staff has determined that since the construction of Units 1 and 2, water levels within the Twin Mountains Formation have fallen below the top of the Twin Mountains Formation in the area of the site, creating a downward gradient and the potential for a downward flow. As a result of these findings, the staff believes that site specific porosity measurements, distances between the bottom of the tanks and Twin Mountains Formation, vertical gradients and tank source terms are different for Units 3 and 4 than for Units 1 and 2.

In order to make its safety determination based on consideration of conservative parameters and alternate scenarios for the transport of accidentally released radioactive liquid effluents, the NRC staff requests that the applicant perform an analysis to determine the impact of vertical migration of an accidental effluent release from Units 3 and 4 to the nearest offsite groundwater receptor within the Twin Mountains Formation. Conservative estimates or measurements of groundwater levels, hydraulic conductivity, effective porosity, flow directions and other hydraulic parameters for the Twin Mountains Formation should be presented and appropriately incorporated into this vertical transport analysis. The applicant is also requested to confirm that receptor concentrations resulting from this analysis comply with Effluent Concentration Limits.

This is supplemental RAI 2.4.13-01-S.

QUESTION NO.: 02.04.13-7 S01

NUREG-0800, Standard Review Plan (SRP), Chapter 2.4.13, 'Accidental Releases of Radioactive Liquid Effluents in Ground and Surface Waters,' establishes criteria that the NRC staff intends to use to evaluate whether an Applicant meets the NRC's regulations.

By letter dated October 5, 2009, the NRC staff issued RAI ID 3673 (RAI No. 116) Question Number 14276 (02.04.13-4), in which the NRC staff asked "Provide a discussion of the assumptions and input parameters, including a table of the assumed undiluted concentration of radionuclides in the tanks at time zero, used with the RATAF code to perform the accidental liquid radioactive effluent release analysis for Comanche peak Nuclear Power Plant, Units 3 and 4 and demonstrate the conservative nature of site-specific parameters in the model input. Please specifically discuss the conservatism of the dilution factor representing the volume of Squaw Creek Reservoir used in the RATAF analysis and the assumed travel time of 365 days."

The applicant responded in document CP-200901565-Log No TXNB-09068-(ML093230229) executed on November 16, 2009. The NRC staff has reviewed the response and has determined that additional information is needed in order to complete its review.

The applicant's response states that this is a DCD related issue and therefore the requested information was not provided to the NRC staff. In a phone call with the US-APWR DCD Applicant on January 20th, 2010, the US-APWR DCD applicant agreed to calculate initial tank concentrations based on 1 percent failed fuel and revise US-APWR DCD Table 11.2-17 to include these concentrations for each tank identified in the table. As such, the COL Applicant is requested to confirm that these revised values were used in the effluent release calculations to calculate concentrations at all receptors identified in the FSAR.

The NRC staff disagrees with the applicant's use of 100 percent instantaneous dilution in the Squaw Creek Reservoir for the horizontal migration scenario since the method does not demonstrate the required level of conservatism. The NRC staff also requests that the applicant present and discuss in the COL application the conservative nature of the value used as the site specific dilution factor.

Using the applicant's parameters and assumptions provided in Table 2.4.12-211 the travel time from the release tank to the Reservoir was estimated by staff to be 189 days for Scenario 2 Pathway 4a. This is more than 10 times faster than the applicant's estimate and much less than the 365 days assumed in the US-APWR DCD generic calculation that the applicant's evaluation references and is dependent upon.

In order to make its safety determination based on consideration of conservative parameters, the NRC staff requests the following information.

1) Provide revised initial concentrations in the tank used in the accidental effluent release analysis and confirm that the tank has highest concentration and volume as required.

2) Explain the conservative nature of the value used as the site specific dilution factor and use conservative site specific estimates of travel times to potential receptors as well as conservative methods to apply the estimates of dilution, where applicable, in the calculation of contaminant concentrations at receptor locations. Sound justifications for the assumptions used in the evaluations should be provided.

3) Provide estimates of contaminant flux into the reservoir from lateral groundwater discharge. This flux information should be used in conjunction with surface water evaluations to determine the concentration and potential exposure through surface water at offsite locations downstream of the Squaw Creek Reservoir Dam.

This is supplemental RAI 2.4.13-03-S.

SUPPLEMENTAL INFORMATION:

This supplemental information is provided to address the NRC Hydrology Open Items 2:4.12-5 and 2.4.13-3 from the audit conducted June 7-9, 2011. During the audit, the NRC identified areas in the site-specific tank failure analysis from FSAR Subsection 2.4.13 that needed clarification or additional information. Specifically, the NRC identified the following areas that could potentially affect the results of the tank failure analysis. The corresponding location in the FSAR that addresses the NRC concerns is provided for ease of review. FSAR mark-ups are included in Attachment 2 for FSAR Subsections 2.4.12 and 2.4.13.

- Address possible runoff from cooling tower areas and potential recharge effect over piping systems that could affect the hydrology model (addressed in revised Subsection 2.4.12.3, Supplemental RAI 147 (Question 02.04.12-8 S01)).
- Surface water recharge into existing fill, which could create a mounding effect of groundwater in the ditches without sufficient time to dissipate (addressed in revised Subsection 2.4.12.3, Supplemental RAI 147 (Question 02.04.12-8 S01)).
- Confirmation that the surface water infiltration to groundwater in the south and west of the plant cutoff will not ultimately affect groundwater in the plant footprint (addressed in revised Subsection 2.4.12.3, Supplemental RAI 147 (Question 02.04.12-8 S01)).
- Confirmation that the groundwater level will not be more than 820 ft. El., and that the hydraulic gradient in the existing is sufficient to transport the contamination to SCR as assumed currently in the horizontal transport calculation (addressed in revised Subsections 2.4.12.3.1 and 2.4.12.5, Supplemental RAI 147 (Question 02.04.12-8 S01)).
- Further justification for using the 25% dilution in the existing fill for the source term transport horizontally to Squaw Creek Reservoir (SCR) and the dilution volume used in SCR prior to reaching the Rotocone device (addressed in revised Subsections 2.4.13.5 and 2.4.13.5.4).
- Consideration that the contamination could not get into the storm water basins (addressed in revised Subsection 2.4.12.3, Supplemental RAI 147 (Question 02.04.12-8 S01).
- Further clarification of the conservative assumptions used in the horizontal transport calculation to SCR (addressed in revised Subsection 2.4.13.5).
- Performance of a vertical calculation to determine transport time to nearest potable water supply through the Glen Rose Formation using a simple calculation that provides the quickest infiltration (addressed in revised Subsections 2.4.12.3.1.1, Supplemental RAI 147 (Question 02.04.12-8 S01) and 2.4.13.4).

Impact on R-COLA

See attached marked-up FSAR Revision 2 pages 2.4-92, 2.4-95, 2.4-96, 2.4-97, 2.4-98, 2.4-99, 2.4-100, 2.4-101, 2.4-102, 2.4-103, 2.4-104, 2.4-105, 2.4-106, 2.4-107, 2.4-108, 2.4-109, 2.4-110, 2.4-111, 2.4-112, 2.4-113, 2.4-114, 2.4-118, 2.4-119, 2.4-120, 2.4-123, 2.4-124, and 2.4-136; and new pages 2.4-259, 2.4-260, and 2.4-261. Some of the mark-ups are editorial in nature and are annotated with a "CTS" number rather than with the RAI question number.

Impact on S-COLA

None; this response is site-specific.

Impact on DCD

None.

SUPPLEMENTAL RESPONSE TO REQUEST FOR ADDITIONAL INFORMATION

Comanche Peak, Units 3 and 4

Luminant Generation Company LLC

Docket Nos. 52-034 and 52-035

RAI NO.: 2929 (CP RAI #22)

SRP SECTION: 02.05.04 - Stability of Subsurface Materials and Foundations

QUESTIONS for Geosciences and Geotechnical Engineering Branch 1 (RGS1)

DATE OF RAI ISSUE: 7/17/2009

QUESTION NO.: 02.05.04-11 S01

NUREG-0800, Standard Review Plan (SRP), Chapter 2.5.4, "Stability of Subsurface Materials and Foundations," establishes criteria that the NRC staff intends to use to evaluate whether an applicant meets the NRC's regulations.

Subsection 2.5.4.5.1.2 in the FSAR proposes that concrete fill will be used for foundation preparing, and further states that the fill concrete has a design compressive strength of 3,000 psi to meet the strength requirement. Please address the concrete durability, as described in American Concrete Institute (ACI) 201.2R, for fill concrete.

Erosion of porous concrete sub-foundation, as described in NRC Information Notice (IN) 97-11, and leaching of calcium hydroxide could be potential problems, since the assumed water ground table (EL. 780 ft) is very close to proposed approximate excavation bottom (about EL. 782 ft), and even could be higher than some localized excavation areas, which need to be deepened below EL. 782 ft to remove disturbed or unstable material. In addition, ground water and perched water seeping down along the sides of the structures could cause potential impact on porous concrete fill. Please explain how the differential settlement due to erosion, and loss of concrete strength due to leaching, will be addressed, and provide justification for the manner in which these potential issues will be addressed.

SUPPLEMENTAL INFORMATION:

This supplemental information is provided in response to NRC Open Items 2.4.12-1 and 2.4.12-3 from the Hydrology Audit of June 7-9, 2011. The response to Question 02.05.04-11 (ML092440357) stated

> The plant structures are equipped with... underground drains to collect underground water and channel it away from the structures.

This supplemental response revises this statement as a result of discussions during the audit.

The open items relate to groundwater levels and the potential transport path of the accidental release of fluids from the Boric Acid Tank. It was anticipated that the underground drainage system would accelerate the transport time of accidental releases to Squaw Creek Reservoir.

During the audit Luminant stated that the underground drainage system will be eliminated, thus eliminating the potentially accelerated transport time for accidental releases. The FSAR has been revised accordingly.

The conclusions of the original response to this RAI are not impacted because elimination of the underground drainage system has no impact on the strength and/or durability of fill concrete since it is replaced by a cementitious membrane coating made out of a crystalline waterproofing compound.

Impact on R-COLA

See attached marked-up FSAR Revision 2 pages 2.5-197, 2.5-202, 2.5-203, 2.5-207, 3.4-1, and 3.4-2.

Impact on S-COLA

None; this response is site-specific.

Impact on DCD

None.

Attachment 2

FSAR Pages Incorporating the Supplemental Responses

73 FSAR pages included:

Pages				Figures
2-lix	2.4-85	2.4-101	2.4-117	2.4.12-212
2-lx	2.4-86	2.4-102	2.4-118	2.4.12-213
2.4-72	2.4-87	2.4-103	2.4-119	2.4.12-214
2.4-73	2.4-88	2.4-104	2.4-120	2.4.12-215
2.4-74	2.4-89	2.4-105	2.4-121	2.4.12-216
2.4-75	2.4-90	2.4-106	2.4-122	2.4.13-201
2.4-76	2.4-91	2.4-107	2.4-123	
2.4-77	2.4-92	2.4-108	2.4-124	
2.4-78	2.4-93	2.4-109	2.4-125	Pages
2.4-79	2.4-94	2.4-110	2.4-136	2.5-197
2.4-80	2.4-95	2.4-111	2.4-241	2.5-202
2.4-81	2.4-96	2.4-112	2.4-259	2.5-203
2.4-82	2.4-97	2.4-113	2.4-260	2.5-207
2.4-83	2.4-98	2.4-114	2.4-261	3.4-1
2.4-84	2.4-99	2.4-115		3.4-2
	2.4-100	2.4-116		

ACRONYMS AND ABBREVIATIONS

CEUS	Central and Eastern United States
CFR	Code of Federal Regulations
cfs	cubic feet per second
CG	cloud-to-ground
CH	Fat Clay
CL	Lean Clay
CO_2	carbon dioxide
COC	Chamber of Commerce
COL	Combined Operating License
COLA	Combined Operating License Application
CoV	coefficient of variation (standard deviation/mean)
cm/sec	centimeters per second
Cp	peaking coefficient
CPNPP	Comanche Peak Nuclear Power Plant
CPSES	Comanche Peak Steam Electric System
CPT	cone penetration test
CSDRS	Certified Site Design Response Spectra
Ct	lag coefficient
CU	Consolidated-Undrained
CWS	Circulating Water System CTS-01353
D	Diameter
DCD	Design Control Document
DF	design factor
DFW	Dallas-Fort Worth
EAB	exclusion area boundary
EGC	Exelon Generation Company

ACRONYMS AND ABBREVIATIONS

Emb	Best Estimate Body-Wave Magnitude
EOF	emergency operation facility
EPA	U.S. Environmental Protection Agency
EPRI	Electric Power Research Institute
EPRI-SOG	Electric Power Research Institute Seismicity Owners
ER	Environmental Report
ESP	Early Site Permit
EST	Earth Science Team
ESW	Essential Service Water CTS_01353
ESWS	essential service water system
ETP	Energy Transfer Partners
ETR	energy transfer ratio
FEMA	Federal Emergency Management Agency
FIRS	Foundation Input Response Spectra
FSAR	Final Safety Analysis Report
ft	feet
g/cc	grams per cubic centimeter
G/G_{max}	dynamic shear modulus reduction
GMRS	Ground Motion Response Spectra
gpd	gallons per day
gpm	gallons per minute
GSI	Geologic Strength Index
HF	high frequencies
HiRAT	High Resolution Acoustic Televiewer
HMR	Hydrometeorological Report
hr	hour

No groundwater was encountered during excavation or construction of CPNPP Units 1 and 2; therefore, there was no dewatering at the site during or after construction of the units (Reference 2.4-214).

2.4.12.2.4 On-Site/Vicinity Groundwater Level Fluctuations

In October 2006, a groundwater investigation was initiated as part of the subsurface study to evaluate hydrogeologic conditions for the CPNPP Units 3 and 4. As part of this groundwater investigation, 47 monitoring wells were installed at 20 locations within the Glen Rose Formation on-site. Figure 2.4.12-208 shows the monitor well locations. Details regarding well construction are presented in Table 2.4.12-208.

Due to the variable nature of groundwater reported at the CPNPP site, the well clusters were installed across CPNPP Units 3 and 4 from west to east of the reactor areas to define the groundwater bearing capabilities and properties of the zones likely to be affected, and to identify the hydraulic connectivity between the zones, if any. Monitoring wells were designated as follows, where XX denotes the well or cluster number for the three zones:

A-zone wells: Regolith or undifferentiated fill monitoring wells (MW-12XXa) were installed if greater than 10 ft of soil was encountered above hollow-stem auger refusal.

B-zone wells: Shallow bedrock monitoring wells (MW-12XXb) were generally completed in the upper 40 to 65 ft of bedrock in an apparent zone of alternating stratigraphy; i.e., claystone, mudstone, limestone, and shale sequences.

C-zone wells: Bedrock monitoring wells (MW-12XXc) were generally completed in deeper bedrock zones consisting of alternating stratigraphy and competent bedrock.

Following well development, water levels were measured from November 2006 to May 2008 (Figure 2.4.12-209) to characterize seasonal trends in groundwater levels. Measured groundwater levels from November 2006 to November 2007 are presented in Table 2.4.12-209. The hydrographs for this groundwater data are presented on Figure 2.4.12-209 and also show precipitation data. The groundwater elevation data is presented by well/cluster location and includes approximate screen elevations for each well in the cluster. In addition, the hydrographs depict rainfall totals for the period of interest. Rainfall data presented was collected from the Opossum Hollow rain gauge located approximately 3.4-mi southwest of the CPNPP Unit 3 and 4 site. Overall, the hydrographs show that water levels in the deeper Glen Rose Formation (C-Zone) do not fluctuate and remain at a constant level near the base of the well or depict a steadily increasing water level, indicating the wells were dry (no groundwater infiltration into the well) or exhibiting slow recharge with the static water level not in equilibrium with the groundwater within the formation. With the exception of seven monitoring wells (MW-1201b, MW-1205b, MW-1207b, MW-1209b, MW-1211b, MW-1212b,and

MW-1217b), hydrographs from the shallow bedrock wells (B-Zone) show a slow and steady increase of water levels over time with little to no fluctuations, also suggesting the static water level within the wells are not in equilibrium with the groundwater within the formation. Available historical information on groundwater and groundwater trends in the Glen Rose Formation is presented in Subsection 2.4.12.2.3.

Water Levels and Potentiometric Elevations in the Regolith (A – Zone)

Groundwater steadily increased from December 2006 to July 2007. Water levels remained constant or decreased slightly from August 2007 to February 2008. Hydrographs from the regolith/fill material wells (A-zone) indicate some slight fluctuations that may be tied to seasonal rainfall. In some of the A-zone wells, there appears to be a slight increase in water levels that may correspond to the spring seasons but there is no significant correlation in the A-zone wells across the site in response to rainfall.

Monitoring well MW-1211a was installed on the northeast portion of CPNPP Units 3 and 4 in undifferentiated fill material. Water levels in this monitoring well were consistent with the normal pool elevation of SCR (775 ft msl) indicating possible hydraulic communication between the former drainage swale and SCR.

Representative potentiometric surface maps for the four quarters (Figure 2.4.12-210 [Sheets 1 through 4]) show that the general shallow (A-Zone) groundwater movement in the vicinity of CPNPP Units 3 and 4 mimics the surface topography, with an apparent groundwater divide along the long axis of the site peninsula. On the northern portion of the peninsula, a northerly flow toward SCR is observed, and a southerly flow toward the Safe Shutdown Impoundment (SSI) is observed on the south side of the site peninsula.

Water Levels and Potentiometric Elevations in the Shallow Bedrock (B – Zone)

Nine of the 16 wells completed in this zone contained no, or negligible, amounts of water for up to eight months before exhibiting measurable water (greater than 1 ft). The majority of these wells exhibited a slow to steady recharge, with no indication of reliable equilibrium conditions over the monitoring period.

Well MW-1211b was installed east of CPNPP Unit 3 in the undifferentiated fill material. During installation, an effort was made to install this well in bedrock; however, due to the thickness and nature of the undifferentiated fill material, the boring was terminated at the bedrock surface (approximately 75 ft below ground surface [bgs]). Water level measurements for this well were consistent with those of regolith monitoring well MW-1211a and the normal pool elevation of SCR over the monitoring period; therefore, the groundwater elevation in monitoring well MW-1211b is not considered to be a measurement of groundwater within the shallow bedrock (B-Zone).

Well MW-1209b was installed northeast of CPNPP Unit 3 in the shallow bedrock below the undifferentiated fill material. Water level measurements for this well were consistent with those of the normal pool elevation of SCR over the monitoring period, showing the shallow bedrock at this location is in communication with SCR; therefore, the groundwater elevation in monitoring well MW-1209b is not considered to be a measurement of groundwater within the shallow bedrock (B-Zone).

Well MW-1212b was installed southeast of CPNPP Unit 3 in the shallow bedrock at the apparent southern extent of the undifferentiated fill material. Water level measurements for this well were approximately 10 feet above the normal pool elevation of SCR over the monitoring period. Due to its location on the southern side of the undifferentiated fill material, which isolates the groundwater in this portion of the site from that in the location of the nuclear islands, the groundwater elevation in monitoring well MW-1212b was not used to determine groundwater flow direction within the shallow bedrock (B-Zone).

Only four shallow bedrock (B-Zone) monitoring wells (MW-1201b, MW-1205b, MW-1207b, and MW-1217b) exhibited consistent water levels, indicating equilibrium conditions. After obtaining static conditions between November 29, 2006, and January 23, 2007, groundwater elevations in these four wells stayed within a 13.76 ft range between 820.08 ft msl (MW-1217b; March 24, 2008) and 833.84 (MW-1215b; October 16, 2007). Monitoring well MW-1217b, located near the center point of CPNPP Unit 3, exhibited the greatest variation following attainment of static conditions, showing water level variations within a 6.97 ft range from January 2007 to May 2008. Comparison with recorded rainfall data at the Opossum Hollow Rain Gage did not show a correlation between water level variations and recorded rainfall data during the monitored period.

Groundwater potentiometric surface maps could not be produced based on only four wells completed in the shallow bedrock (B-Zone) that exhibited consistent equilibrium conditions and evidence that the groundwater within the shallow bedrock is recharged from the perched groundwater within the overlying soils. However, the groundwater levels within the four wells show a general groundwater gradient trend towards SCR and it is expected that the groundwater potentiometric surface will follow that of the overlying soils.

Water Levels and Potentiometric Elevations in the Bedrock Monitoring Wells (C - Zone)

Of the 14 groundwater monitoring wells screened in bedrock, six contained no, or negligible, amounts of water over the monitoring period and eight exhibited a slow to steady recharge, with no indication of reliable equilibrium conditions.

Groundwater potentiometric surface maps could not be produced due to the lack of reliable groundwater, or evidence of non-equilibrium conditions within the deeper C-Zone monitoring wells.

~~Based on the above mentioned observations, groundwater at the CPNPP 3 and 4 site appears to be limited to a perched interval within the overlying soils on top of the weathered upper Glen Rose Formation limestone (upper bedrock). Based on the lack of reliable groundwater within the bedrock beneath the site soils, groundwater availability decreases significantly with depth.~~ From site observations, it is concluded that the groundwater within the regolith recharges the weathered, upper portions of the bedrock, with little infiltration to deeper bedrock zones. RCOL2_02.04.12-8 S01

Groundwater beneath the CPNPP Units 3 and 4 occurs in two zones, separated by the Glen Rose Limestone aquitard. The uppermost zone is perched water residing in the surficial soils and uppermost weathered Glen Rose limestone bedrock. As stated previously, the groundwater found in the uppermost bedrock is attributed to recharge from the overlying soils and is transient, based on precipitation amount. The next zone occurs in the Twin Mountains Formation, beneath the Glen Rose limestone aquitard. This zone is the nearest "permanent" groundwater source with potentiometric surfaces well below the elevation of the building foundations on site. RCOL2_02.04.12-8 S01

Groundwater flow direction within the regolith is toward SCR. Flow direction of groundwater within the upper bedrock (groundwater B-Zone) appears to flow eastward toward SCR. However, based on the limited groundwater availability within the bedrock, depicted by long-term, non-equilibrium water levels within most bedrock monitoring wells, groundwater flow within the upper bedrock is limited and likely linked to flow within the overlying perched groundwater.

2.4.12.2.5 Aquifer Characteristics

Groundwater has been identified within the undifferentiated fill, regolith and bedrock beneath the CPNPP Units 3 and 4 sites; therefore, this subsection provides characteristics of these zones. During construction, the undifferentiated fill material and regolith are expected to be removed in the power block area. The foundation elevation is estimated to be approximately 782 ft msl on the bedrock. Groundwater currently measured in the soil zones (undifferentiated fill material and regolith) and the Glen Rose Formation is considered "perched" and will be removed during construction activities. Characteristics of the Glen Rose Formation indicate that it is not a groundwater bearing unit and a permanent dewatering system will not be required.

2.4.12.2.5.1 Porosity

Soil Zones

The soils occurring on the CPNPP site are described in the Hood and Somervell counties soil survey information provided by the USDA Natural Resources Conservation Service's on-line Soil Data Mart website (Reference 2.4-259). A total of 18 soil mapping phases representing 17 soil series occur within the CPNPP site boundary. Descriptions of each soil series are provided in Table

2.4.12-210 and the location of the soil mapping phases are shown on Figure 2.4.12-211.

The two soil types mapped in the vicinity of the CPNPP Units 3 and 4 build areas include the Tarrant – Bolar association and Tarrant – Purves association. Physical properties for these soil types indicate clay content ranges of 20 to 60 percent, moist bulk densities of 1.10 gram per cubic centimeter (g/cc) to 1.55 g/cc, saturated hydraulic conductivities between 4.2 x 10-5 centimeters per second (cm/sec) and 1.4 x 10-3 cm/sec, and available water capacities of 0.05 inch per inch (in/in) to 0.18 in/in (Reference 2.4-260).

The site is underlain by a sedimentary rock sequence of the Glen Rose Formation which, at the surface, has been weathered to a clayey, silty, sandy overburden soil with some rock fragments (referred to as regolith). However, most of the CPNPP site is situated in areas disturbed by previous construction activities associated with the construction of CPNPP Units 1 and 2. Porosity in the undifferentiated fill or regolith materials was evaluated based on the grain size distributions from the current investigation:

- Undifferentiated Fill - Based on the grain size distribution of the on-site soils, the total porosity was determined by averaging the porosity range for sand, silt, and clay. The average total porosity of the on-site regolith and undifferentiated fill is assumed to be 0.45. Based on a lack of information regarding effective porosity in the undifferentiated fill, an effective porosity of 0.45 was assumed.

- Regolith – As mentioned above, the average total porosity of the on-site regolith and undifferentiated fill/regolith (soils) is assumed to be 0.45. To estimate the effective porosity of the on-site soils, the arithmetic mean of the effective porosities for fine grained sand, silt, and clay were averaged (Reference 2.4-261). The average effective porosity of the on-site regolith and undifferentiated/regolith is assumed to be 0.20.

Bedrock Zones

The bedrock is comprised of limestone from the Glen Rose Formation. The results of the geotechnical analysis performed at the CPNPP Units 3 and 4 site indicated that an average total porosity of the shallow bedrock (limestone and shale) is 25.6 percent and the average total porosity of limestone is 11.9 percent. The Argonne National Laboratory publication, Data Collection Handbook to Support Modeling Impacts of Radioactive Material in Soil, dated April 1993 (Reference 2.4-261) references an arithmetic mean of the effective porosity for limestone of 14 percent. Consequently, the most conservative approach when determining velocity and travel time is to use the measured 11.9 percent porosity value which provides a higher calculated velocity through the shallow bedrock.

2.4.12.2.5.2 Permeability

The permeability of a material is a measure of the ability to transmit water. To assist in determining permeability of the Glen Rose Formation, forty packer-pressure tests were performed in five test borings at 5-foot intervals of varying depth at CPNPP Units 3 and 4 in 2007. The results of these packer tests indicated little to no water take into the Glen Rose Formation; therefore, the formation is essentially impermeable. Detailed examination of cores from test borings revealed minor solutioning features and minimal fractures. Drill water occasionally was lost while drilling through the upper weathered zone and is believed to have occurred at the soil/bedrock interface.

Due to very slow groundwater recharge, single well slug tests were performed on six CPNPP Units 3 and 4 monitoring wells using the Bouwer & Rice method in April 2007. Of the six wells tested, two were screened in the regolith, one was screened in an undifferentiated fill/regolith zone, and three were screened in the shallow bedrock zone. Hydraulic conductivity for the wells screened in the regolith or undifferentiated fill/regolith zone ranged from 2.93×10^{-5} cm/s to 5.00×10^{-4} cm/s. Hydraulic conductivity for the wells screened in the shallow bedrock ranged from 6.29 x 10-6 cm/s to 1.37×10^{-5} cm/s. RCOL2_2.4.12-8 S01

To investigate groundwater communication with SCR, test well RW-1 was installed in an area of undifferentiated fill within a former drainage swale on the northeast portion of the CPNPP Units 3 and 4 site. A step test and 72-hr pumping test were performed on test well RW-1 in April 2007. The step test was performed to determine the pumping rate for the 72-hr pumping test. Data for the step test and 72-hr pumping test were analyzed using the Cooper-Jacob Step Test and Theis Recovery Test methods. The results of the 72-hr pump test estimated hydraulic conductivity at 1.70×10^{-3} cm/s during pumping and 3.5×10^{-3} cm/s during recovery.

2.4.12.3 Subsurface Pathways

~~Subsurface pathways include the unsaturated zones and saturated zones beneath the CPNPP Units 3 and 4. Groundwater is the primary transport mechanism for possible liquid effluent release. Groundwater movement and velocity will vary depending on the matrix through which it flows. The rate of flow (i.e. the velocity) of groundwater depends on (1) the hydraulic conductivity and porosity of the medium through which it is moving and (2) the hydraulic gradient. Higher groundwater velocities occur with greater hydraulic conductivity and hydraulic gradient.~~ RCOL2_02.04.12-8 S01

~~It is assumed that a release from either unit would first encounter the engineered fill surrounding the A/B and R/B. This engineered fill material is connected to the fill surrounding various site systems, but in particular to the ESW piping tunnels and UHS basins, since these are embedded at an equal depth as the A/B and R/B (Figure 2.4.12-212). Portions of the engineered fill surrounding these systems are~~

~~in contact with the existing fill to the east of Unit 3 and to the north of Unit 4; therefore, a release from the unit will flow within the engineered fill until it comes in contact with the existing fill. As stated in Subsection 2.4.12.2.4, the existing fill is in communication with SCR and has a higher hydraulic conductivity; therefore, groundwater within the engineered fill surrounding the A/B and R/B will be drained through the contact with the existing fill into SCR. As the hydrogeologic properties of the engineered fill are unknown at this time, the groundwater transport time through the engineered fill will be considered negligible and any release will be conservatively assumed to begin at the engineered fill/existing fill boundary closest to SCR.~~ RCOL2_02.04.12-8 S01

~~Due to very slow groundwater recharge, single well slug tests were performed on six monitoring wells using the Bouwer & Rice method in April of 2007 at CPNPP Units 3 and 4. Of the six wells tested, two were screened in the regolith, one was screened in an undifferentiated fill/regolith zone, and three were screened in the shallow bedrock zone. Hydraulic conductivity for the wells screened in the regolith or undifferentiated fill/regolith zone ranged from 2.93×10^{-6} cm/s to 5.00×10^{-4} cm/s. Hydraulic conductivity for the wells screened in the shallow bedrock ranged from 6.29×10^{-6} cm/s to 1.37×10^{-5} cm/s.~~

~~A step test and 72-hr pumping test were performed on aquifer pump test well RW-1 in April of 2007. To investigate groundwater communication with SCR, pump test well RW-1 was installed in an area of undifferentiated fill within a former drainage swale on the northeast portion of CPNPP Units 3 and 4. The step test was performed to determine the pumping rate for the 72-hr pumping test. Data for the step test and 72-hr pumping test were analyzed using the Cooper-Jacob Step Test and Theis Recovery Test methods. The results of the 72-hr pump test estimated hydraulic conductivity at 1.70×10^{-3} cm/s during pumping and 3.5×10^{-3} cm/s during recovery.~~

~~Due to site grading activities during plant construction, maximum groundwater elevations within the plant site will be limited to the invert elevation of the southern and western drainage trench, which has a maximum elevation of 820 ft msl. Recharge to the upper bedrock zone in the plant site will be restricted by drainage into this trench; therefore limiting the maximum conservative groundwater elevation in the plant site to 820 ft. msl.~~

~~Soil distribution characteristics for radiological isotopes (i.e., Co_{60}, Cs_{137}, Fe_{55}, I_{129}, Ni_{63}, Pu_{239}, Tc_{99}, U_{235}) were determined from soil and water samples collected along the preferred groundwater flow path. This data is discussed in detail in Subsection 2.4.13 to assist in the development of transport calculations for fate and transport analyses in the event of accidental releases of effluents to groundwater.~~ The groundwater system in the vicinity of CPNPP Units 3 and 4 will be modified following construction activities. Soils and weathered bedrock will be removed from the area surrounding Units 3 and 4 during site grading activities, exposing fresh bedrock surfaces of the Glen Rose limestone. Excavations and trenches will be incised in fresh bedrock to accommodate the various building

structures and piping. Granular engineered fill will be placed in excavations and trenches during construction and covered with a low permeability cap to limit surface water infiltration directly into the engineered fill. Descriptions of groundwater flow pathways in piping trenches included in this discussion, such as those pertaining to the ESW and CWS, describe groundwater within the engineered fill surrounding these subsurface structures. RCOL2_02.04.12-8 S01

The engineered fill has not been designed, so the hydraulic properties of the fill are unknown at this time. However, it is anticipated the fill will be a compacted granular fill with porosity and hydraulic conductivity far exceeding that of the surrounding limestone bedrock. Groundwater is expected to travel rapidly through connected areas of engineered fill to surrounding areas of existing fill with little recharge to the underlying and surrounding limestone bedrock.

Post-construction groundwater flow paths will be influenced most by the placement of engineered fill. Structural excavations are approximately the same depth and will provide preferential pathways for the movement of groundwater from Units 3 and 4 subgrade structures to surrounding existing fill and SCR.

A conceptual post-construction groundwater model is depicted in Figure 2.4.12-216. The majority of the rainfall onsite will move via surface runoff into stormwater drainage ditches that discharge into stormwater retention basins located east of Unit 3 (eastern retention basin) and west of Unit 4 (western retention basin). Surface runoff from the cooling tower area will be directed to the western retention basin. Overflow from these retention basins discharges to SCR. A low permeability cap (not yet designed) is expected to be placed atop all engineered fill areas limiting the infiltration of surface water from site runoff and stormwater ditch accumulations. Although these low permeability caps will be designed to prevent infiltration, it is possible for surface water seepage into the engineered fill to occur.

Rainwater will infiltrate into the native soils to the west and south of Units 3 and 4 and will provide some recharge into the underlying limestone bedrock. However, because of the low permeability of the Glen Rose limestone, infiltration of groundwater from the surficial soils or from direct exposure to rainfall into the shallow limestone will be limited. The limited bedrock infiltration could result in lateral seepage of groundwater into the engineered fill surrounding the subsurface structures at the CPNPP site.

Although the engineered fill has not been designed, it can be assumed the compacted granular fill placed in the excavated areas will be of significantly higher hydraulic conductivity than the surrounding limestone bedrock. Groundwater seepage into the engineered fill will flow along the piping trenches and building annuli toward the existing fill east of Unit 3 and north of Unit 4, along the banks of SCR. East of Unit 3, portions of the engineered fill surrounding the CWS and ESW piping trench are in contact with existing fill ($K_h = 3.5 \times 10^{-3}$ cm/sec). Similarly, north of Unit 4, portions of the engineered fill surrounding the UHS

basins and ESW piping trench are in contact with existing fill (K_h = 5.0 x 10^{-4} cm/sec.) Groundwater within the existing fill materials east of Unit 3 and north of Unit 4 are in direct communication with SCR and the groundwater elevations within these materials are in equilibrium with the SCR pool elevation. Therefore, these existing fill areas will drain groundwater from the engineered fill into SCR. RCOL2_02.04.12-8 S01

2.4.12.3.1 Groundwater Pathways

~~Although the discussions of groundwater movement is a reasonable scenario for groundwater flow, it is assumed that the actual groundwater is subject to three-dimensional control structures (horizontal, vertical, and any secondary porosity that may be present) and does not have uniform flow across the site.~~ RCOL2_02.04.12-8 S01

~~Groundwater pathways are considered from the CPNPP Unit 3 and 4 Auxiliary Buildings where the boric acid tank (BAT) is located, to SCR, which is the nearest potential receptor.~~

~~Placement of engineered fill surrounding the A/B, R/B, ESW piping, UHS basins, and circulating water piping will affect the direction and flow rate of groundwater infiltrating from the remaining bedrock. Portions of the engineering fill surrounding these subsurface structures are in communication with the existing fill on the site (Figure 2.4.12-212). The existing fill is in communication with SCR, and due to the low hydraulic conductivity of the bedrock, it is expected that groundwater infiltrating into the engineered fill will migrate through the engineered fill into the existing fill and then enter SCR, with little to no groundwater transport through the upper bedrock. Since the geohydrologic properties of the engineered fill are unknown at this time, groundwater transport time through the engineered fill is conservatively assumed to be negligible.~~

~~Two postulated groundwater pathway scenarios, CPNPP Unit 3 to SCR through the existing fill east of CPNPP Unit 3, and CPNPP Unit 4 to SCR through the existing fill north of CPNPP Unit 4, represent the most conservative pathways from a two-reactor site where groundwater flow is possibly in different directions from each unit (Figure 2.4.12-212). Both flow paths utilize a conservative, straight line flow path approach from the point of release and the shortest distance and highest measured hydraulic conductivity for the pathway assessed. A straight line flow path is considered the most conservative as the actual groundwater pathways are expected to be tortuous, resulting in longer transport times and hydraulic conductivities (K_h) that are expected to be lower than the highest measured.~~

~~To estimate groundwater travel time through the existing fill, the effective porosity of the site soil (0.20 from Subsection 2.4.12.2.5.1) is used as a conservative estimate. As post construction groundwater levels within the existing fill are unknown, groundwater elevation within the existing fill is conservatively assumed to be at the maximum expected groundwater level of 820 ft msl. The normal operating pool elevation for SCR is 775 ft. msl; however, the minimum operating~~

~~SCR pool elevation of 770 ft msl is used to produce the highest conservative hydraulic gradient.~~ RCOL2_02.04.12-8 S01

~~The swale east of Unit 3 was filled with the excavation debris from Units 1 and 2; thus, it is considered to be a haphazard mélange of clay through boulder size material with some debris present. The swale north of Unit 4 appears to have been constructed in a more methodical manner to support building foundations. Construction data for the swale fills are not available. However, it is assumed the fill properties are sufficiently different to allow the conservative use of the individual hydraulic conductivities from each swale fill testing in the groundwater pathway analysis based on the following:~~

- ~~evidence from visual observations~~
- ~~data obtained from the geotechnical drilling program~~
- ~~results of the pump and slug test analysis performed on monitoring wells within the individual existing fill materials~~
- ~~there is no connection between the two filled areas~~
- ~~the appearance of different placement methods and dates of the swale fill materials.~~

~~For the groundwater velocity and travel time assessment described below, the groundwater pathway 1 hydraulic conductivity (K_h), measured from observation well RW 1 recovery test (3.50×10^{-3} cm/s) represents the hydraulic conductivity measured in the existing fill east of Unit 3. The groundwater pathway 2 K_h, measured from monitoring well MW 1219a slug testing (5.00×10^{-4} cm/s) represents the hydraulic conductivity measured in the existing fill north of Unit 4.~~

~~For groundwater pathway 1 (Figure 2.4.12-213), it is assumed that an instantaneous release from the BAT would travel out of the CPNPP Unit 3 A/B into the engineered fill surrounding the A/B and R/B. It would then travel to the closest engineered/existing fill interface, located to the east of the Unit 3 turbine building. For conservatism, it is assumed that the transport time to the fill interface will be negligible. It will then travel 600 ft through the existing fill to the closest release location in SCR. The travel time from the release point to SCR via the existing fill east of Unit 3 is conservatively estimated at 145 days.~~

~~For groundwater pathway 2 (Figure 2.4.12-214), it is assumed that an instantaneous release from the BAT would travel out of the CPNPP Unit 4 A/B into the engineered fill surrounding the A/B and R/B. It would then travel to the closest engineered/existing fill interface, located to the north of the CPNPP Unit 4 UHS basin. For conservatism, it is assumed the transport time to the fill interface will be negligible. It will then travel 350 ft through the existing fill to the closest release~~

~~location in SCR. The travel time from the release point to SCR via the existing fill north of Unit 4 is conservatively estimated at 346 days.~~ RCOL2_02.04.12-8 S01

~~Groundwater gradients, velocities, and travel times are summarized in Table 2.4.12-211.~~

~~Cross sections depicting the post-construction groundwater flow pathways from CPNPP Unit 4 to SCR are presented in Figure 2.4.12-214.~~

~~The current soil and rock material comprising the hydrologic A zone (undifferentiated fill and regolith) and B zones (shallow bedrock) discussed in Subsection 2.4.12.2.4 will be removed for construction of plant foundations, resulting in the removal of the perched groundwater from the power block area. Post construction surface water infiltration to the Glen Rose Formation limestone will be reduced with the construction of surface water impoundments and an improved drainage system throughout the CPNPP Units 3 and 4 site. The grading and drainage plan and placement of engineered fill material are designed to preclude surface water infiltration into the limestone on which the foundation will be constructed.~~

~~Based on the excavation of the perched zones in the A zone and B zones in power block area, the impermeable nature of the Glen Rose Formation, and the absence of any water wells producing from the Glen Rose Formation in the CPNPP Units 3 and 4 site area, impact to present and projected groundwater users is not anticipated. The postulated groundwater pathway scenarios discussed in this subsection and further in Subsection 2.4.13, project SCR to be the nearest receptor. Evaluation of the accident effects of a contaminant release to groundwater from CPNPP Units 3 and 4 is discussed in detail in Subsection 2.4.13.~~ Groundwater is the primary transport mechanism for possible liquid effluent release. The velocity of groundwater depends on the hydraulic conductivity and porosity of the medium through which it is moving and the hydraulic gradient.

Higher groundwater velocities occur with greater hydraulic conductivity and hydraulic gradient. It is assumed that a release from either unit would first encounter the engineered fill that has been placed in limestone excavations surrounding the A/B and R/B. The engineered fill material connects directly to other engineered fill material also placed in limestone excavations and/or trenches that surround various site systems. For example, the ESW piping trenches and UHS basins are embedded at an equal depth as the A/B and R/B (Figure 2.4.12-212). Portions of the engineered fill surrounding these systems are in contact with the existing fill to the east of Unit 3 and to the north of Unit 4. Therefore, a release from the unit will flow within the engineered fill until it comes in contact with the existing fill. As stated in Subsection 2.4.12.2.4, the existing fill is in communication with SCR and has a higher hydraulic conductivity than the surrounding limestone. Therefore, groundwater within the engineered fill surrounding the A/B and R/B will be drained through contact with the existing fill into SCR. As the hydrogeologic properties of the engineered fill are unknown at this time, the groundwater transport time through the engineered fill is considered

negligible and any release is conservatively assumed to begin at the engineered fill/existing fill boundary closest to SCR. RCOL2_02.04.12-8 S01

The extent of engineered fill placement is depicted on FSAR Figure 2.4.12-212. This figure shows the engineered fill within the CWS piping trench along the southern boundary of the plant site will underlie much of the southern cut bank. The CWS piping trench is shown between the western cut bank and the higher topographic areas to the west of the plant site. A significant portion of the CWS piping trench east of the Unit 3 turbine building is depicted as being in contact with the existing fill to the east of Unit 3. Due to their down gradient position and high permeability, the existing fills east of Unit 3 and north of Unit 4 will provide a drainage location for the engineered fill.

The projected groundwater movement in the vicinity of the Units 3 and 4 power block was assessed to evaluate contaminant migration for the postulated release scenario (Subsection 2.4.13). For the release scenario, radwaste contaminant sources include the Units 3 and 4 boric acid tanks (BATs), located at the lowest level in the A/B. For the assessment of alternative pathways, five locations were assumed to be plausible points of exposure (i.e., locations at which groundwater would be discharged to the surface to allow human contact or to facilitate transport). The pathways evaluated are:

Path #1: Unit 3 through existing fill to SCR

This scenario postulates groundwater flow released from the engineered fill/existing fill interface east of Unit 3 to the nearest location along SCR (Figures 2.4.12-212 and 2.4.12-213). Discharge would be directly to the water of SCR with a travel distance of 600 ft. The released contamination would then travel in the surface waters of SCR.

Path #2: Unit 4 through existing fill to SCR

This scenario postulates groundwater flow released from the engineered fill/existing fill interface north of Unit 4 to the nearest location along SCR (Figures 2.4.12-212 and 2.4.12-213). Discharge would be directly to the water of SCR with a travel distance of 350 ft. The released contamination would then travel in the surface waters of SCR.

Path #3: Units 3 or 4 through the Glen Rose Limestone to Twin Mountains Formation Aquifer

This scenario postulates a hypothetical groundwater flow pathway from a BAT release into the engineered fill surrounding the Unit 3 or 4 A/B, then migrating vertically to the upper contact between the Twin Mountains Formation and the Glen Rose Formation (Figure 2.4.12-216). Due to the higher hydraulic conductivity provided by Paths #1 and #2, only a portion of the released BAT volume is assumed to migrate vertically. Discharge of the vertical portion would be directly to the groundwater within the Twin

Mountains Formation aquifer with a minimum vertical travel distance of 193 ft. The released contamination would then travel within the groundwater of the Twin Mountains Formation aquifer. RCOL2_02.04.12-8 S01

Path #4: Unit 3 to the eastern stormwater retention basin

This scenario postulates groundwater flow released from the engineered fill east of the Unit 3 ESW piping trench through the Glen Rose limestone to the eastern retention basin. Discharge from the retention basin would be directly to the water of SCR. The released contamination would then travel in the surface waters of SCR.

Path #5: Unit 4 to the western stormwater retention basin

This scenario postulates groundwater flow released from the engineered fill west of the Unit 3 EWS piping trench through the Glen Rose limestone to the western retention basin. Discharge from the retention basin would be directly to the water of SCR. The released contamination would then travel in the surface waters of SCR.

Paths #4 and #5 to the stormwater retention basins are considered to be implausible because their overflow elevations will be less than 812 ft msl for the basin east of Unit 3 or 810 ft msl for the basin west of Unit 4 (very low hydraulic gradients) and they are separated from the engineered fill by a minimum of 150 feet of low permeability Glen Rose Limestone. Additionally, due to the limitation of the maximum groundwater elevation to 813.5 ft msl, and the low permeability of the Glen Rose formation, groundwater within the engineered fill will not migrate into the drainage ditches surrounding Units 3 and 4.

Although these pathways for groundwater movement are reasonable scenarios for groundwater flow, it is assumed that the actual groundwater is subject to three-dimensional control structures (horizontal, vertical, and any secondary porosity that may be present) and will not have uniform flow across the site.

Placement of engineered fill surrounding the A/B, R/B, ESW piping trench, UHS basins, and CWS piping trench will affect the direction and flow rate of groundwater infiltrating from the remaining bedrock. Portions of the engineering fill surrounding these subsurface structures are in communication with the existing fill on the site (Figure 2.4.12-212). The existing fill is in communication with SCR and due to the low hydraulic conductivity of the bedrock, it is expected that groundwater infiltrating into the engineered fill will migrate through the engineered fill into the existing fill and then enter SCR, with little to no groundwater transport through the upper bedrock. Since the hydrogeologic properties of the engineered fill are unknown at this time, groundwater transport time through the engineered fill is conservatively assumed to be negligible.

2.4.12.3.1.1 Groundwater Travel Times

RCOL2_02.04.12-8 S01

Groundwater within the engineered fill is conservatively assumed to be limited to a maximum elevation of 813.5 ft msl (Subsection 2.4.12.5); however, for the purpose of calculation groundwater velocity and travel times for the following pathways, groundwater elevation within the existing fill is assumed to be saturated to 820 ft msl, which is above the conservatively assumed maximum groundwater elevation of 813.5 ft msl.

Paths #1 and #2

Since post-construction groundwater levels within the existing fill are unknown, groundwater elevation within the existing fill is assumed to be saturated to 820 ft msl, which is above the conservatively assumed maximum groundwater elevation of 813.5 ft msl. The normal operating pool elevation for SCR is 775 ft. msl; however, the minimum operating SCR pool elevation of 770 ft msl is used to produce the highest conservative hydraulic gradient.

The swale east of Unit 3 was filled with the excavation debris from Units 1 and 2; thus, it is considered to be a haphazard mélange of clay to boulder-size material with some debris present. The swale north of Unit 4 appears to have been constructed in a more methodical manner to support building foundations. Construction data for the swale fills are not available, but it is assumed the fill properties are sufficiently different to allow the conservative use of the individual hydraulic conductivities in the groundwater pathway analysis based on the following:

- evidence from visual observations
- data obtained from the geotechnical drilling program
- results of the pump and slug test analysis performed on monitoring wells within the individual existing fill materials
- there is no connection between the two filled areas
- the appearance of different placement methods and dates of the swale fill materials.

For the groundwater velocity and travel time assessment described below, the hydraulic conductivity (Kh) for Path #1, measured from observation well RW-1 recovery test (3.50×10^{-3} cm/s), represents the hydraulic conductivity measured in the existing fill east of Unit 3. The Kh for Path #2, measured from monitoring well MW-1219a slug testing (5.00×10^{-4} cm/s), represents the hydraulic conductivity measured in the existing fill north of Unit 4.

As stated in Subsection 2.4.12.2.4, the groundwater present in the existing fill appears to be in communication with SCR. Therefore, it is assumed that any

groundwater infiltrating into the existing fill (from the engineered fill, seepage from the underlying bedrock, or from surface precipitation infiltration) will rapidly drain along the bedrock surface below the existing fill until it reaches the equivalent elevation of SCR while the existing fill above the pool elevation of SCR essentially remains unsaturated (Figures 2.4.12-213 and 2.4.12-214). Since the exact nature of the groundwater flow within the existing fill is difficult to quantify, it is assumed for conservatism that the engineered fill is saturated to a maximum groundwater elevation of 820 ft msl and has a linear hydraulic gradient to the nearest shoreline of SCR. This assumption produces the steepest hydraulic gradient and fastest transport time through the existing fill. RCOL2_02.04.12-8 S01

The expected actual groundwater flow in the saturated portion of the existing fill below the surface elevation of SCR will be dependent on the operating elevation of SCR. The SCR normal pool elevation is 775 ft msl with a minimum pool elevation of 770 ft. msl. When the SCR pool elevation is rising, surface water is expected to infiltrate and recharge the existing fill causing a rise in the groundwater elevation. Conversely, when the SCR pool elevation is falling, the groundwater from the existing fill will discharge into SCR causing the groundwater elevation within the existing fill to lower. During static pool conditions, it is expected that slight discharges from the existing fill into SCR will occur due to recharge from the engineered fill, seepage from the underlying bedrock, or from surface precipitation infiltration. Recharge into the existing fill from these sources is expected to be minor in volume. Consequently, the assumptions regarding the conservative groundwater hydraulic gradient method described in the previous paragraph would produce faster travel times from the release point to the shoreline of SCR as compared to the expected slow flushing of the existing fill.

For groundwater Path #1 (Figure 2.4.12-213), it is assumed that a release from the Unit 3 BAT would instantaneously travel out of the Unit 3 A/B into the engineered fill surrounding the A/B and R/B. For conservatism, it is assumed that this release would travel as a slug without any dilution within the groundwater present in the engineered fill. It would then travel through the engineered fill of the Unit 3 R/B and A/B and the ESW piping trench to the closest engineered/existing fill interface, located to the east of the Unit 3 turbine building (Figure 2.4.12-212). Since the hydraulic properties for the engineered fill have not yet been determined, it is assumed that the transport time to the fill interface will be negligible and no retardation or dilution of the released Unit 3 BAT contents is credited. At the assumed point of release, the released Unit 3 BAT contents will enter the existing fill east of Unit 3. The geometry and properties of the existing fill/engineered fill contact are presently unknown; therefore, for conservatism, it is assumed the contact will not retard any groundwater or radionuclide flow from the engineered fill into the existing fill. The released Unit 3 BAT contents will then travel 600 ft through the existing fill to the closest shoreline location on SCR. The travel time from the release point to SCR via the existing fill east of Unit 3 is conservatively estimated at 145 days.

For groundwater Path #2 (Figure 2.4.12-214), it is assumed that a release from the Unit 4 BAT would instantaneously travel out of the Unit 4 A/B into the

engineered fill surrounding the A/B and R/B. For conservatism, it is assumed that this release would travel as a slug without any dilution within the groundwater present in the engineered fill. It would then travel through the engineered fill of the Unit 4 R/B and A/B and the ESW piping trench to the engineered/existing fill interface located to the north of the CPNPP Unit 4 between the UHS basins. Although this is not the closest existing fill/engineered fill contact, it is the existing fill/engineered fill contact closest to the shoreline of SCR and therefore, would have the shortest distance to travel through the existing fill. Since the hydraulic properties for the engineered fill have not yet been determined, it is assumed the transport time to the fill interface will be negligible and no retardation or dilution of the released Unit 4 BAT contents is credited. At the assumed point of release, the released Unit 4 BAT contents will enter the existing fill north of Unit 4. The geometry and properties of the existing fill/engineered fill contact are presently unknown; therefore, for conservatism, it is assumed the contact will not retard any groundwater or radionuclide flow from the engineered fill into the existing fill. The released Unit 4 BAT contents will then travel 350 ft through the existing fill to the closest shoreline location on SCR. The travel time from the release point to SCR via the existing fill north of Unit 4 is conservatively estimated at 346 days. RCOL2_02.04.12-8 S01

Path #3

To estimate groundwater travel time through the underlying bedrock to the Twin Mountains Formation (Path #3), the effective porosity of the Glen Rose limestone (0.119 from Subsection 2.4.12.2.5.1) is used as a conservative estimate. As post-construction groundwater levels within the existing fill are unknown, groundwater elevation within the existing fill is conservatively assumed to be at 820 ft msl.

For groundwater Path #3 (Figure 2.4.12-215), it is assumed that an instantaneous release from the BAT would travel out of the CPNPP Units 3 or 4 A/B into the engineered fill surrounding the A/B and R/B. Although vertical groundwater movement is considered improbable due to thickness and extremely low hydraulic conductivity of the lower Glen Rose limestone, a hypothetical pathway was evaluated using a higher hydraulic conductivity (1×10^{-6} cm/s) than those measured during site investigations. For conservatism, it was assumed this pathway would allow downward advective groundwater flow from the engineered fill material surrounding the A/B to the Twin Mountains Formation. The travel time from Units 3 or 4 to the Twin Mountains Formation via the postulated vertical pathway is conservatively estimated at 6858 days (18.78 years).

Groundwater gradients, velocities, and travel times are summarized in Table 2.4.12-211.

Cross-sections depicting the post-construction groundwater flow pathways from CPNPP Units 3 and 4 to SCR and the Twin Mountains Formation are presented in Figures 2.4.12-213, 2.4.12-214, and 2.4.12-215.

Impact to present and projected groundwater users is not anticipated based on the excavation of the perched zones in the A-zone and B-zones in the power block area, the impermeable nature of the Glen Rose Formation, and the absence of any water wells producing from the Glen Rose Formation in the CPNPP Units 3 and 4 site area. The postulated groundwater pathway scenarios discussed in this subsection and further in Subsection 2.4.13 project SCR to be the nearest plausible groundwater release location to surface water. Evaluation of the accident effects of a contaminant release to groundwater from CPNPP Units 3 and 4 is discussed in detail in Subsection 2.4.13. RCOL2_02.04.12-8 S01

Based on this evaluation, Path #1 would be the most conservative groundwater pathway as it has the fastest travel time to SCR.

2.4.12.3.2 Nearby Groundwater Users

While no use of groundwater at the CPNPP site is planned, consideration is given for the movement of groundwater beneath the site because of pumping. Potable-use wells at CPNPP are completed in the Twin Mountains Formation, a confined aquifer below the impermeable Glen Rose Formation. Most domestic wells in the area are completed in the Twin Mountains Formation (Table 2.4.12-212). The on-site wells completed in the Twin Mountains Formation are not considered capable of reversing groundwater flow beneath the CPNPP Units 3 and 4 site. There are no domestic or public water supply wells within a 0.5-mi. radius of the site that are completed in the Glen Rose Formation. (Figure 2.4.12-204). No off-site wells are considered capable of reversing groundwater flow beneath the site, or vice versa, based on the geographic positions of these wells (i.e., the distance of the domestic wells from the power block area and their completion in the Twin Mountains Formation). RCOL2_02.04.12-8 S01

2.4.12.4 Monitoring or Safeguard Requirements

Accident effects are discussed in Subsection 2.4.13 and the radiation protection program is discussed in Section 12.5. Additionally, analysis of the relationship of the CPNPP groundwater to seismicity and the potential for related soil liquefaction and the potential for undermining of safety-related structures is discussed in Section 2.5. A groundwater monitoring program will be developed before fuel load that will include radiological sampling based upon post-construction configuration.

2.4.12.5 Site Characteristics for Subsurface Hydrostatic Loading

According to the Design Control Document (DCD) for the US-APWR, the design maximum groundwater elevation is 1 ft below plant grade. The CPNPP plant grade elevation is 822 ft msl; therefore, the design maximum groundwater elevation is 821 ft msl relative to the current elevation of the Glen Rose Formation. The Glen Rose Formation is an impermeable limestone that confines the groundwater in the underlying Twin Mountains Formation aquifer. Not all of the wells completed in the Glen Rose Formation were sampled; however, the wells that were sampled and purged, purged dry and water did not return for several

days to weeks. All deep Glen Rose wells have been reported as "dry" or reported with less than 1-foot of water. The Twin Mountains Formation is at least ~~230~~185 ft below the ~~Glen Rose Formation~~excavation depths of the Units 3 and 4 subgrade structures; therefore, a dewatering system will not be required during construction and the installation and operation of a permanent dewatering system is not planned. ~~A dewatering system will not be required during construction. Normal construction practices will be employed to remove water from seepage and rainfall.~~ Based on the documented low primary and secondary permeability of the Glen Rose Formation limestone at the site, and the fact that the engineered fill placed around the CPNPP Units 3 & 4 reactor and auxiliary buildings will be well compacted, it is unlikely that a seismic event would cause a seismically-induced rise in groundwater elevations (Reference 2.4-298).

RCOL2_02.04.12-8 S01

RCOL2_2.4.12-09 S01

~~Based on the removal of the soil overlying the bedrock surrounding the site foundations, and the maximum groundwater elevation within the engineered fill constrained by the southern and western trench drain to less than 820 ft. msl, the design maximum groundwater elevation is expected to be satisfied.~~

RCOL2_02.04.12-8 S01

The current soil and rock material comprising the hydrologic A-zone (undifferentiated fill and regolith) and B-zones (shallow bedrock) discussed in Subsection 2.4.12.2.4 will be removed for construction of plant foundations, resulting in the removal of the perched groundwater from the power block area. Post-construction surface water infiltration to the Glen Rose Formation limestone will be reduced with the construction of surface water impoundments and an improved drainage system throughout the CPNPP Units 3 and 4 site. The grading and drainage plan and placement of engineered fill material are designed to minimize surface water infiltration into the limestone on which the foundation will be constructed.

The extent of engineered fill placement is depicted on Figure 2.4.12-212. This figure shows the engineered fill within the CWS piping trench along the southern boundary of the plant site will underlie much of the southern cut bank. The CWS piping trench is shown between the western cut bank and the higher topographic areas to the west of the plant site. A significant portion of the CWS piping trench east of the Unit 3 turbine building is also depicted as being in contact with the existing fill to the east of Unit 3. Due to its downgradient position, the existing fill will provide a drainage path for the engineered fill.

The CWS piping trenches are embedded to the approximate depth of other subgrade structures and any horizontal seepage of groundwater along bedding planes from the south or west will intersect the CWS piping trenches prior to reaching the engineered fill surrounding the A/B or R/B and then be transported within the engineered fill to the existing fill. The engineered fill properties have not been designed, so the nature of the engineered/existing fill contact is unknown at this time.

The hydraulic conductivity of the existing fill east of Unit 3 has been measured at 3.5x10-3 cm/sec or 74.2 gallons/day/ft^2 (gpd/ft^2). The hydraulic conductivity of the existing fill north of Unit 4 has been measured at 5x10-4 cm/sec or 10.6 gpd/ft^2. RCOL2_02.04.12-8 S01

The easternmost contact surface between the CWS piping trench engineered fill and the existing fill located east of Unit 3 will be at least 350 ft wide (Figure 2.4.12-212) and 23.5 ft deep (assuming the maximum groundwater elevation at 813.5 ft msl minus elevation of the top of the piping trench fill concrete at 790 ft msl). Monitoring well MW-1214a (B2028) located near this contact location reported the top of rock elevation at 781 ft msl; therefore, this entire contact surface will be within the eastern engineered fill. This results in a contact surface area between the existing and engineered fill at that location of at least 8225 ft^2. This would be considered a minimum contact surface as excavation and stabilization requirements for installation of the CWS piping would most likely require additional excavation and placement of engineered fill or leveling concrete.

Based on the hydraulic conductivity of the existing fill (74.2 gpd/ft^2) located east of Unit 3, the drainage rate of the fully saturated fill at the expected maximum groundwater elevation (813.5 ft msl) would be 610,295 gpd (424 gpm) through the existing/engineered fill contact at this location. Even at the minimum hydraulic conductivity (10.6 gpd/ft^2), this would allow a drainage rate of approximately 87,185 gpd (60.54 gpm).

Due to the unknown orientation, size, and properties of the contacts between the engineered fill and existing fill for Units 3 and 4, this assessment considers the drainage rate for only a small portion of the total engineered/existing fill contact surfaces present. The actual drainage rate is expected to be significantly higher due to additional contact surfaces not considered in this assessment. Given the estimated drainage rates of the existing fill areas, it is not expected that precipitation recharge into the southern or western soils, drainage pathways from the adjacent cooling tower area, or infiltration from the stormwater ditches will exceed the potential drainage rate provided by the existing fill. However, for conservatism, the groundwater travel time and velocity calculations for the existing fill (Subsection 2.4.12.3) assume the groundwater elevation will build up to a level of 820 ft msl, above the highest (most conservative) trench drain elevation (813.5 ft msl) where surface seepage will occur through the base of the trench drain, limiting the maximum water level with the engineered fill to 813.5 ft msl. Brief mounding above 813.5 ft may occur due to short term stormwater accumulation in the drainage trenches; however, the mounding would not exceed the tank failure analysis elevation (820 ft msl) or the DCD maximum allowable elevation (821 ft msl).

Based on this assessment, it is unlikely that preferential seepage from native soils overlying limestone bedrock to the west and south of Units 3 and 4 would cause a rise in groundwater elevation within the engineered fill that would exceed the maximum allowed groundwater elevation of 821 ft msl as defined by the DCD.

Furthermore, the existing release scenarios described in Subsection 2.4.13 include the most conservative engineered/existing fill contact locations. Therefore, all other engineered/existing fill contact locations would result in longer groundwater travel pathways through the existing fill to SCR and do not affect the results of the conservative accident analysis presented in Subsection 2.4.13. RCOL2_02.04.12-8 S01

Based on the removal of the soil overlying the bedrock surrounding the site foundations, and the maximum groundwater elevation within the engineered fill constrained by the southern and western trench drain to less than 813.5 ft. msl, the design maximum groundwater elevation is expected to be satisfied.

Based on the documented low primary and secondary permeability of the Glen Rose Formation limestone at the site, and the fact that the engineered fill placed around the CPNPP Unit 3 and 4 R/B and A/B will be well-compacted, it is unlikely that a seismic event would cause a seismically-induced rise in groundwater elevations (Reference 2.4-298).

2.4.13 Accidental Releases of Radioactive Liquid Effluent in Ground and Surfacewaters

---

CP COL 2.4(1) Add the following at the end of the DCD Subsection 2.4.13.

Historical and projected groundwater flow paths were evaluated in Subsection 2.4.12 to characterize groundwater movement from the nuclear island area to a point of exposure. Figure 2.4.12-203 depicts subsurface conditions that control the movement of groundwater beneath the CPNPP Unit 3 and 4 site. Based on groundwater flow directions (Figure 2.4.12-209, Sheets 1, 4, 7, and 10), different flow paths are applicable from Units 3 and 4 via horizontal groundwater movement to the nearest surfacewater body (SCR). Subsection 2.4.12 provides the locations and users of surface water in the CPNPP site area.

A conceptual model of radionuclide transport through groundwater to the nearest surfacewater body is described below. The conceptual model and alternate conceptual model developed consider both vertical and horizontal radioactive liquid effluent transport based upon the post-construction configuration of CPNPP Units 3 and 4 (see Figures 2.4.12-212 through 2.4.12-214).

2.4.13.1 Identification of Source Term and Soil/Water Distribution of Liquid Effluent

In performing the evaluation of Postulated Radioactive Releases Due to Liguid-Containing Tank Failures, the following tanks were considered in determining which tank would have the highest concentration and the largest volume of radionuclides:

Holdup Tank - located in the Auxiliary Building (A/B), a Seismic Category II building.

Waste Holdup Tank - located in the A/B

Boric Acid Evaporator - located in the A/B

Boric Acid Tank - located in the A/B

Volume Control Tank - located in the Reactor Building (R/B)~~,~~ a Seismic Category I Building~~.~~ CTS-01351

Auxiliary Building Sump Tank - located in the A/B

Reactor Building Sump Tank - located in the R/B

Primary Makeup Water Tank - located outside

Refueling Water Storage Auxiliary Tank - located outside

Chemical Drain Tank - located in the A/B

The Volume Control Tank, the Chemical Drain Tank, and Sump Tanks were eliminated from consideration based on smaller volumes and lower radionuclide contents than the Boric Acid Tank (BAT). The Primary Makeup Water Tank was eliminated from consideration based upon the fact that the Primary Makeup Water Tank stores demineralized water from the Treatment System. and low level radioactive condensate water from the Boric Acid Evaporator. Condensate water contains low levels of radionuclide concentrations. including tritium. Additionally, the Refueling Water Storage Auxiliary Tank (RWSAT) was eliminated from consideration because it stores refueling water. Prior to refueling, tank water is supplied to the refueling cavity where the reactor coolant radionuclide concentration dilutes with refueling cavity water. Radionuclide concentration of cavity water is reduced by the purification system of the Chemical and Volume Control System (CVCS) and the Spent Fuel Pit Cooling and Purification System (SFPCS) during refueling operations. Upon refueling completion, part of the cavity water is returned to this tank where the radionuclide concentration is low. Accordingly. the impact of RWST or Primary Makeup Water Storage Tank failure is small.

After eliminating the tanks described above, the remaining tanks left to consider for the failure analysis are those in the A/B, which is a seismic category II Building. As shown in DCD Figure 1.2-29, these tanks are located on the lowest elevation of the A/B at elevation 793 ft ms. In selecting the appropriate tank for the failure analysis, the guidance in Branch Technical Position (BTP) 11-6 was utilized based upon the concentrations generated from the RATAF Code for Pressurized Water Reactors. The concentration of the radioactive liquid in the tanks, such as the Boric Acid Evaporator, the Holdup Tank, and the BAT, are larger than the Waste Holdup Tank since they receive reactor coolant water extracted from the Reactor Coolant System. Since the enrichment factor of 50 is considered for the liquid phase of the Boric Acid Evaporator. the radioactive concentrations in the liquid phase of the Boric Acid Evaporator, and in the BAT (which receives the enriched liquid from the Boric Acid Evaporator) becomes large when compared to the other tanks. The BAT has been selected since its volume is larger than the liquid phase of the Boric Acid Evaporator. Credit is taken for the removal effect by demineralizers or other treatment equipment for the liquid radioactive waste prior to entering the tank. No chelating agents are used in the plant system design in order to provide chemical control of the reactor-coolant. Only a very small amount of chelating agents is used in the sampling system for analysis. The sampling drain, which contains only a small amount of chelating agents is directly sent to the dedicated chemical drain tank and treated separately. Chemical agents used in laboratory analysis are also sent to the chemical drain tank for treatment. Therefore, neither the chelating agents nor the chemical agents used in the sampling analysis will have any effect on the transport characteristics of the source term liquid effluent release analysis.

The source term concentrations considered for these tanks are identified in DCD Table 11.2-17, and were calculated using NUREG-0133 and the RATAF Code for Pressurized Water Reactors. The BAT is located in the northeast (NE) corner of the A/B (see DCD Figure 12.3-1). The A/B basemat elevation is at approximately 785 ft msl. The BAT elevation is expected to be at 793 ft msl. Ground level at the site is expected to be at 822 ft msl. The BAT contained the largest concentration and volume of radionuclides that was closest to the effluent concentration limits (ECLs). Isotope concentrations less than 1.0×10^{-3} in fraction of concentration limits are excluded from the evaluation. Since credit cannot be taken for liquid retention by unlined building foundations, it is assumed that 80 percent of the content of the tank is released to the environment, consistent with the guidance in BTP 11-6, March 2007.

While groundwater functions as the transport media for fugitive radionuclides, interaction of individual radionuclides with the soil matrix delays their movement. The solid/liquid distribution coefficient, K_d, is, by definition, an equilibrium constant that describes the process wherein a species (e.g., a radionuclide) is partitioned by adsorption between a solid phase (soil) and a liquid phase (groundwater). Soil properties affecting the distribution coefficient include the texture of soils (sand, loam, clay, or organic soils), the organic matter content of the soils, pH values, the soil solution ratio, the solution or pore water concentration, and the presence of competing cations and complexing agents. Because of its dependence on many soil properties, the value of the distribution coefficient for a specific radionuclide in soils can range over several orders of magnitude under different conditions. The measurement of distribution coefficients of radionuclides within the preferential groundwater pathways allows further characterization of the rate of movement of fugitive radionuclides in groundwater.

The site-specific K_d coefficients were selected based upon radionuclides listed in 10 CFR Part 20, Appendix B, Table 2. Three soil borings were chosen for sampling characteristics. Soil and groundwater samples were collected from monitoring wells MW-1201 (located southwest of the CPNPP Unit 4 nuclear island), MW-1208 (located east of the Unit 3 nuclear island), and MW-1219 (located northeast of the Unit 4 nuclear island) (Figure 2.4.12-207). Soil samples from each monitoring well were collected, based on the availability of recovered soils, at depths ranging from approximately 18 to 54 feet below ground surface. Dry wells exhibiting very slow recharge, and the aquifer testing observations wells were not considered for sampling. Soil boring samples gathered from the two hydraulically upgradient wells and hydraulically downgradient wells were submitted to Argonne National Laboratory for analysis of the radionuclides listed in FSAR Section 2.4.13 based upon the radionuclides listed in 10 CFR Part 20, Appendix B and those radionuclides that would be expected to exist in the tanks were considered for the failure analysis. The soil boring samples were submitted for laboratory analysis of soil distribution characteristics for specific radiological isotopes (i.e., Co-60, Cs-137, Fe-55, I-129, Ni-63, Pu-242, Sr-90, Tc-99, U-235). Results of the K_d analyses are presented in Table 2.4.13-201.

Since the A/B is where the BAT, the Holdup Tank and the Waste Holdup Tanks are to be located at Units 3 and 4, appropriate values were evaluated for "nuclides of interest" (Table 2.4.13-201) based on transport to SCR without retardation or retention through subsurface media. Thus, using the conservative transport time analysis, and considering nuclide decay times, those nuclides which could be expected to challenge 10 CFR Part 20, Appendix B, concentration limits were considered. The BAT was selected as the tank that had the greatest volume and largest concentration of radionuclides, where credit is taken for removal equipment and demineralizer beds. The purpose of the K_d analysis was to estimate the potential migration of accidental releases from the footprint areas of the proposed new units. The K_d results presented in Table 2.4.13-201 indicate that the radionuclides would be delayed in their movement through the groundwater pathway to SCR. The tank failure analysis assumed no distribution of contaminants (no K_d coefficients used) based upon the site-specific hydrogeological characteristics. ~~It is conservatively assumed that the contaminants would transport along the groundwater pathway horizontally to SCR without retardation or retention in the subsurface media, and that there would be no groundwater dilution prior to reaching SCR.~~ RCOL2_02.04.13-05, 06, 07 S01

2.4.13.2 Development of Alternate Conceptual Model and Site-Specific Geological and Hydrogeological Parameters

~~The~~Utilizing the groundwater pathways described in Subsection 2.4.12.3.1 and Figures 2.4.12-212 through 2.4.12-215, alternative conceptual models were used to determine a bounding set of plausible groundwater flow paths by considering the nearest surface water body, SCR, current groundwater elevations measured in wells near the proposed power block area, the measured pool elevation of SCR (gradient to the SCR), horizontal seepage of groundwater along bedding planes from the south or west, and a conservative pathway from a postulated release point to SCR. RCOL2_02.04.13-05, 06, 07 S01 RCOL2_02.04.13-05, 06, 07 S01

After exploring alternative transport pathways, two plausible pathways were determined to bound potential release pathways. Refer to Figure 2.4.13-212 and associated cross section Figures 2.4.12-213 and 2.4.12-214 for the horizontal release pathways. The vertical release pathway is implausible and improbable based upon site-specific hydrogeology discussed in Subsection 2.4.12; however, a hypothetical release of a small volume of BAT source term contamination is evaluated ~~Vertical release pathways are eliminated from consideration as discussed~~ in Subsection 2.4.13.4. The vertical release pathway is shown in Figure 2.4.12-215. Alternate horizontal groundwater pathways from each unit moving south or west from the BAT A/B location were eliminated from consideration as this movement would be away from SCR and would not be consistent with the hydraulic gradients for the area surrounding the CPNPP Units 3 and 4 shown on Figure 2.4.12-210, Sheets 1 through 4. RCOL2_02.04.13-05, 06, 07 S01

CPNPP Units 3 and 4 are to be constructed on the Glen Rose Formation. The Glen Rose limestone is essentially impermeable, ranging from 217 to 271 ft thick, and is underlain by the Twin Mountains Formation, which contains the first aquifer

beneath the site. Figures 2.5.5-202 and 2.5.5-203 provides a generalized cross section of the pre-construction site conditions. Figures 2.4.12-213 and 2.4.12-214 show the post-construction pathway cross-sections for the shortest distance releases to SCR via horizontal groundwater pathways. The groundwater flow pathways were developed based on groundwater measured in monitoring wells in the CPNPP Unit 3 and 4 plant area and measured elevations in SCR. Wells were installed across the site in zones to define the groundwater bearing capabilities and properties of the zones, and identify the hydraulic connectivity between the zones, if any. The well zones are defined as A-Zone (regolith or undifferentiated fill material), B-Zone (shallow bedrock) and C-Zone (deeper bedrock) and are described in Subsection 2.4.12.2.4. RCOL2_02.04.13-05, 06, 07 S01

The process used to develop alternative conceptual models of groundwater flow included the following:

- Groundwater flow pathways were developed based on groundwater measured in monitoring wells in the Units 3 and 4 plant area, measured elevations in SCR, surface topography, and observed water levels over time.

- Groundwater measured in all three zones was considered perched based on measurements. Groundwater in the A-zone regolith was attributed to surface water infiltration. Groundwater measured in the undifferentiated fill near SCR was attributed to SCR.

- Groundwater in the B-zone was not continuous across the site Non-equilibrium conditions and the reported dry wells in the B-zone wells indicated that the groundwater was perched. Groundwater located in fill areas near SCR was found to be in communication with SCR.

- Negligible groundwater was gauged in the C-zone wells, representing essentially dry conditions. Consequently, this zone was not considered a groundwater bearing unit.

- Post-construction section configuration of the A/B building, the Ultimate Heat Sink (UHS) cooling tower structure area and other structures were used in identifying the bounding set of plausible pathways. In addition to Figures 2.4.12-213 and 2.4.12-214 horizontal pathway cross sections and Figure 2.4.12-215 for hypothetical vertical release pathway, the following site plan views and section plans were utilized in identifying the bounding set of plausible pathways: RCOL2_02.04.13-05, 06, 07 S01

  - Site Plan View Figure 1.2-1R;

  - Power Block at Elevation 793" ft msl Plan View Figure 1.2-2R;

  - ESW Pipe Tunnel Sectional View A-A' Figure 1.2-202; and

- Ultimate Heat Sinks A and B Sectional Views Figure 1.2-206.

- Rainfall infiltration effect on the liquid effluent and plausible release pathway is also considered based upon post-construction structures and building configurations. Rainfall infiltration is not considered a contributing factor affecting the source term release pathway. No dilution effects of groundwater or rainfall are considered in the liquid effluent release analysis. Rainfall infiltration effects are discussed in Subsections 2.4.12.3 and 2.4.13.3. RCOL2_02.04.13-05, 06, 07 S01

2.4.13.3 Potential Effects of Construction on Groundwater Flow Paths

The current soil and rock material comprising the hydrologic A-zone (undifferentiated fill and regolith), and the B-zone (shallow bedrock) will be removed for construction of plant foundations, resulting in the removal of the perched groundwater from the plant area. Post-construction surface water infiltration to the Glen Rose Formation limestone will be reduced with the construction of surface water impoundments and an improved drainage system throughout the Units 3 and 4 site. The grading and drainage plan and placement of engineered fill material are designed to preclude surface water buildup near the plant foundation, reducing the possibility of surface water infiltration into the limestone on which the foundation will be constructed.

During construction, the undifferentiated fill material and regolith will be removed in the power block area, and replaced with engineered fill material. A dewatering system will not be used but rainfall and seepage will be removed during construction.

In October 2006, a groundwater investigation was initiated as part of the subsurface study to evaluate hydrogeologic conditions for CPNPP Units 3 and 4. As part of this groundwater investigation, 47 monitoring wells were installed at 20 locations within the Glen Rose Formation onsite. Due to the variable nature of groundwater reported at the CPNPP site, the well clusters were installed across the footprint of CPNPP Units 3 and 4 from west to east of the reactor areas to define the groundwater bearing capabilities and properties of the zones likely to be affected, and to identify the hydraulic connectivity between the zones, if any. Following well development, water levels were measured from November 2006 to May 2008 to characterize seasonal trends in groundwater levels.

Rainfall data presented was collected from the Opossum Hollow rain gauge located approximately 3.4-mi southwest of the CPNPP Unit 3 and 4 site. Hydrographs were developed and are presented in Figure 2.4.12-209. These hydrographs show that water levels in the deeper Glen Rose Formation (C-zone) do not fluctuate and remain at a constant level near the base of the well or depict a steadily increasing water level, indicating that this water is not actual groundwater. Hydrographs from the shallow bedrock wells (B-zone) show a slow and steady increase of water levels over time with little to no fluctuations, also suggesting water levels are related to infiltration from the overlying soils and not

actual groundwater. Hydrographs from the regolith/fill material wells (A-zone) indicate some slight fluctuations that may be tied to seasonal rainfall. In some of the A-zone wells there appears to be a slight increase in water levels that may correspond to the spring seasons but there is no significant correlation in the A-zone wells across the site in response to rainfall.

The water levels in the regolith/fill material and the upper zone of the Glen Rose Formation (A-zone and B-zone, respectively) were attributed to surface run-off and were not a true measure of permanent groundwater in the formation. Groundwater steadily increased from December 2006 to July 2007. Water levels remained constant or decreased slightly from August 2007 to February 2008.

Nine of the 16 wells completed in Shallow Bedrock (B – Zone) contained no, or negligible, amounts of water for up to eight months before exhibiting measurable water (greater than 1 ft). The majority of these wells exhibited a slow to steady recharge with no indication of reliable equilibrium conditions over the monitoring period.

Of the 14 groundwater monitoring wells screened in Bedrock (C-Zone), six contained negligible ~~to~~ amounts of water over the monitoring period and eight exhibited a slow to steady recharge with no indication of reliable equilibrium conditions. CTS-01351

The complete discussion of rainwater infiltration effects is provided in Subsection 2.4.12.3 with additional information regarding stormwater overflows to retention basins described in Subsection 2.4.12.3.1. Site characteristics for subsurface hydrostatic loading and effect on groundwater pathways is discussed in Subsection 2.4.12.5. RCOL2_02.04.13-05, 06, 07 S01

The Grading and Drainage Plan shown on Figure 2.4.2-202 was developed based upon the effects of local intense precipitation, as discussed in Subsection 2.4.2.3, and aids in moving precipitation away from structures and buildings considered in the plausible pathways for the liquid effluent release analysis.

Rainfall infiltration is not considered a contributing factor affecting the source term release pathway. No dilution effects of groundwater or rainfall are considered in the liquid effluent release analysis.

2.4.13.4 Vertical Release Pathway ~~Elimination~~

RCOL2_02.04.13-05, 06, 07 S01

This subsection evaluates two scenarios: (1) eliminating the vertical release pathway based upon site-specific hydrogeology, which is the most probable and plausible scenario (Subsection 2.4.13.4.1); and (2) the hypothetical case where a small volume of the BAT source term contamination is released through the Glen Rose Formation reaching the Twin Mountains Formation aquifer with ultimate transport through the aquifer out the CPNPP property boundary to a potential unrestricted potable water supply well (Subsection 2.4.13.4.2).

The hypothetical scenario uses input parameters that are not indicative of site-specific measured parameters. Using input parameters that would allow vertical migration to occur, this evaluation is performed to demonstrate the duration required for this migration pathway. The BAT source term contamination released to the environment is chemically tritiated acidic water with some dissolved and un-dissolved metallic particulate. Similar chemical characteristics have been found at NRC-licensed decommissioning sites (Haddam Neck, Maine Yankee, Yankee Rowe, to name a few) in outdoor tanks that had been leaking for many years. Regardless of the site-specific hydrology or geology, utilizing substantial sampling and analysis in accordance with NRC guidance, it was observed that similar chemical constituents readily disperse with existing groundwater nearby the genesis of the source term contamination leak. Therefore, the BAT source term contamination is expected to behave similarly to groundwater and readily disperse within the groundwater. Furthermore, the metallic particulate will actually disperse within the engineered fill around the BAT cubicle base mat and the primary contaminates that readily flow with existing groundwater are tritium, strontium and cesium. This hypothetical scenario demonstrates the long duration (81.53 years) for the source term contamination to reach a potential unrestricted potable water supply well. This long duration is beyond the life of the plant with license extension and it is expected that the contamination would be readily removed prior to migrating through the Glen Rose Formation. Thus, this hypothetical scenario is not considered plausible or probable. RCOL2_02.04.13-05, 06, 07 S01

2.4.13.4.1 Vertical Release Pathway Elimination

Both SCR and the CPNPP Units 1 and 2 restricted potable water supplies wells were considered as receptors. The CPNPP Units 1 and 2 potable water supply wells are restricted access potable water supply wells completed in the Twin Mountains Formation aquifer and approximately 1990 feet south and downgradient of the CPNPP Unit 3 A/B. The nearest unrestricted potable water supplies completed in the Glen Rose Formation are approximately 4 miles south of the CPNPP Unit ~~CPNPP~~ 3 A/B. and the nearest unrestricted potable water supply wells completed in the Twin Mountains Formation is approximately 1 mi west of the CPNPP Unit ~~CPNPP~~ 4 A/B (FSAR Subsection 2.4.12.3.2 and Figures 2.4.12-204 and 2.4.12-206). The restricted potable water supply wells in Units 1 and 2 (Figure 2.4.1-213) were not considered as possible receptors based upon the following: CTS-01351 CTS-01351 CTS-01351

The BAT is at elevation 793 ft msl, while the Auxiliary Building basemat elevation is at 785 ft msl. Because the Auxiliary Building is a Seismic Category II Building, it is assumed that a crack will form in the building during a beyond design basis seismic event or some other physical phenomena, and the radioactive liquid would travel vertically into the surrounding formation. At this basemat elevation of 785 ft msl, the hydrogeologic formation is in the deeper portion of the Glen Rose Formation, which consists primarily of impermeable limestone. For the release to reach the Twin Mountains Formation, which is approximately 150 feet below the Glen Rose Formation, the liquid release would have to travel completely through the Glen Rose Formation. Vertical migration pathways are considered improbable RCOL2_02.04.13-05, 06, 07 S01

due to the thickness (approximately 150 ft) and extremely low hydraulic conductivity of the lower Glen Rose limestone:

- Packer tests in the power block areas show low hydraulic conductivities (10^{-8} to 10^{-9} cm/sec range, or no water takes) from plant grade elevation (822 ft msl) to 677 ft msl (Table 2.5.4-206).

- Transport of contaminates through formations with hydraulic conductivities less than 10^{-6} cm/sec is controlled by diffusion rather than advection (Reference 2.4-295)

- ~~Units 1 and 2 utilized diffusion for contaminant movement and assumed no groundwater transport.~~ RCOL2_02.04.13-05, 06, 07 S01

- Discrete engineering layers in the Glen Rose formation can be traced in the subsurface throughout the site and correlated approximately 2000 feet away in the CPNPP Units 1 and 2 borings and historical excavation photographs.

- Known post-construction excavation limits can be correlated with the stratigraphy exposed in the Glen Rose formation photographs.

A complete discussion of the core borings stratigraphy and CPNPP Units 1 and 2 historical excavation photographs as compared to CPNPP Units 3 and 4 borings is provided in Subsection 2.5.4.3.1.

The closest CPNPP Units 1 and 2 potable water supply well is approximately 1.25 miles away (Figure 2.4.1-213) from either the CPNPP Unit 3 or Unit 4 A/B (Figure 2.4.12-208). The liquid release would be in the Glen Rose formation, which at the level of the BAT is essentially impermiable to groundwater flow. Because the vertical migration pathway was considered implausible, the only plausible release scenario would involve a horizontal release to SCR. Therefore, the alternate conceptual models chosen were to transport the liquid radioactive release through the engineered fill and undifferentiated fill/regolith pathway to SCR (as described in Subsection 2.4.12.3.1 and shown on Figures 2.4.12-212 through 2.4.12-214). CTS-01351

2.4.13.4.2 Hypothetical Vertical Release

RCOL2_02.04.13-05, 06, 07 S01

Although vertical groundwater movement is considered improbable due to the thickness and extremely low measured hydraulic conductivity of the lower Glen Rose Formation limestone (Subsection 2.4.12.3.1.1), a hypothetical vertical flow path (Pathway #3 on Figure 2.4.12-216) is assumed with a higher hydraulic conductivity than that measured during site investigations. For conservatism, it is assumed this pathway would allow downward advective groundwater flow from the engineered fill material surrounding the A/B through the limestone Glen Rose Formation ultimately reaching the Twin Mountains Formation aquifer.

Path #3 postulates groundwater flow from a BAT release into the engineered fill surrounding CPNPP Units 3 or 4 A/B, then migrating vertically through the underlying limestone bedrock to the upper contact between the Twin Mountain Formation and the Glen Rose Formation. Due to the higher horizontal hydraulic conductivity provided by Paths #1 and #2 (Subsection 2.4.13.5), only a portion (105.6 gallons out of the total 52,800 gallons released in the bounding horizontal pathway to SCR) of the source term contamination is assumed to migrate vertically. Discharge of the small vertical volume would be directly to the groundwater within the Twin Mountain Formation aquifer with a minimum vertical travel through the Glen Rose Formation to the Twin Mountain Formation of 193 feet. The released source term contamination would then travel within the Twin Mountain Formation aquifer up-gradient approximately 0.75 miles (3960 ft.) or down-gradient approximately 2.6 miles (13,728 ft.) until it exits the CPNPP property boundary where it could be potentially available for use in an unrestricted potable water supply well. RCOL2_02.04.13-05, 06, 07 S01

A hypothetical release is from the subsurface of the A/B entering the engineered fill placed around the R/B and A/B during construction. Geohydrologic properties of the engineered fill are not known at this time; therefore, transport time and retardation properties are unknown and considered negligible for this evaluation.

The basis for the conservative assumptions utilized in the hypothetical scenario follows. The conservative assumptions include:

- Groundwater elevations were chosen to produce the highest hydraulic gradients, resulting in the highest groundwater flow velocity through the Glen Rose limestone.

- Groundwater velocities are calculated assuming the engineered fill surrounding the A/B is fully saturated to 820 ft msl (which is above the conservatively assumed groundwater elevation of 813.5 msl) to the upper surface of Twin Mountain Formation (592 ft msl).

- Although vertical groundwater movement is considered improbable due to the thickness and extremely low hydraulic conductivity of the lower Glen Rose Formation, a hydraulic conductivity of 1 E-06 cm/sec is chosen that would allow for advective (Darcy's equation) vertical migration. Locally, the Glen Rose Formation is considered an aquitard which forms an upper boundary to the Twin Mountain Formation aquifer. Packer tests (Table 2.5.4-206) in the power block areas show low hydraulic conductivities ranging from E-08 to E-09 cm/sec (or no water takes) from plant grade elevation (822 ft msl) to 677 ft msl.

- Once the source term contamination reaches the Twin Mountain Formation aquifer after 18.78 years migration through the Glen Rose Formation, it continues to flow up-gradient for 62.75 years or down-gradient for 217.55 years before exiting the CPNPP property boundary where a potential unrestricted water supply well could be placed. The

shorter duration of flow time of 62.75 years up-gradient is conservatively chosen for the hypothetical scenario.

RCOL2_02.04.13-05, 06, 07 S01

- A straight line flow path through the Glen Rose limestone is considered most conservative as the actual groundwater pathway is much more tortuous. Also; transport times would be much longer and hydraulic conductivities much lower.

- No dilution by Glen Rose formation groundwater is credited and only 200,000 gallons or 7.571 E08 ml out of the entire Twin Mountain Formation aquifer (millions of gallons) is credited for diluting the source term contamination in the groundwater.

The vertical migration through the Glen Rose Formation was assessed using the Darcy equation for groundwater flow velocity:

$V_V = (K_V \times I_V) / \eta$ Equation 1

Where:

V_V = vertical groundwater flow velocity, ft/day

K_V = vertical hydraulic conductivity, ft/day

I_V = vertical hydraulic gradient, unitless

η = effective porosity (ft/ft), unitless

The vertical hydraulic gradient is calculated by:

$I_V = (E_h - E_L) / (E_B - E_L)$ Equation 2

Where:

E_h = highest groundwater elevation (perched aquifer), ft above msl

E_B = elevation of the reactor building base mat, ft above msl

E_L = lowest groundwater elevation (top of Twin Mountain Formation), ft above msl

Vertical groundwater travel time through the Glen Rose Formation is calculated by:

$T_V = (L_V / V_V) / 365.25$ Equation 3

Where:

T_V = vertical groundwater travel time, years

L_V = vertical distance, (E_B - E_L), ft

RCOL2_02.04.13-05, 06, 07 S01

Input parameters to determine time through Glen Rose Formation:

- The average effective porosity (η) of the Glen Rose limestone is estimated at 11.9% (Subsection 2.4.12.2.5.1).
- Vertical distance is from the top of the A/B base mat elevation (E_B = 785 ft msl; Subsection 2.4.13.1) to the contact elevation of the Glen Rose/Twin Mountains Formations (E_L = 592 ft msl; Subsection 2.4.12.1.2).
- Highest groundwater elevation (E_H) (perched aquifer) is 820 ft msl.
- KV of the Glen Rose limestone is conservatively assumed to be the lowest hydraulic conductivity where Darcy's equation (advection) is valid for groundwater transport (1 E-06 cm/sec).

Using the input parameters, the vertical hydraulic gradient (I_V) is calculated to be 1.18. The vertical groundwater velocity (V_V) through the Glen Rose Formation limestone is 0.028 ft/day. At this groundwater velocity, it takes approximately 18.78 years to reach the Twin Mountain Formation upper cap. The groundwater travel time to Twin Mountain Formation is extremely conservative as it assumes a straight-line path; actual pathways will be long and tortuous and would add to the time duration.

Travel times through the Twin Mountains aquifer either up-gradient or down-gradient is calculated considering the following information:

Groundwater elevation data was obtained from six nearby USGS wells completed in the Twin Mountain Formation aquifer with recorded water levels in 2010, including the CPNPP water supply well (USGS well 3240604). Reported groundwater elevations for the five identified wells and the CPNPP water supply well are shown in Table 2.4.13-212 (TWDB, 2011a and TWDB, 2011b).

These water levels were plotted to produce a potentiometric surface map of the Twin Mountain Formation in the vicinity of the CPNPP. Groundwater hydraulic gradients were calculated based on the distance between USGS wells 3242403 (702.58 ft msl) and 3243805 (394.40 ft msl). These two well locations bound the CPNPP site and are in the approximate gradient direction of groundwater flow within the Twin Mountain Formation.

Average hydraulic conductivity (Kh) of the Twin Mountain Formation was reported as 9 ft/day (Reference 2.4-299). Porosity of the sandstone samples retrieved during the 2007 CPNPP pre-COL application investigation was reported as

ranging between 19 to 37% with an average value of 27% (Subsection 2.5.4.2.3.1.3).

RCOL2_02.04.13-05, 06, 07 S01

Using Darcy's equation and the input parameters below for groundwater flow in a granular media, the groundwater velocity was calculated to be 0.17 ft/day:

1. Well 3242403 - 702.58 ft msl
2. Well 3243805 - 394.40 ft msl
3. Distance Between Wells - 59460 ft
4. Hydraulic Gradient - 0.0052 unitless
5. Hydraulic Conductivity - 9.00 ft/day
6. Porosity - 0.27 unitless

Groundwater Travel Time in the Twin Mountains Formation

Nearest Property Boundary

The distance to the nearest property line from the CPNPP is approximately 0.75 miles (3960 ft) southwest (FSAR Figure 2.1-205). This would be the location of the closest point where unrestricted water well could be drilled in the vicinity of the CPNPP. Using an average of 365.25 days per year (including leap years) a travel time of approximately 62.75 years is obtained for a release at the Glen Rose/Twin Mountain Formation contact below the CPNPP to reach the nearest potential location of an uncontrolled groundwater user.

Although this is the nearest property boundary to the CPNPP Units 3 and 4, this location is in a hydraulically up-gradient position to a release at the site and any release from the site would actually travel down-gradient for greater distances to reach an uncontrolled location; therefore, this is considered a conservative analysis.

Nearest Down-Gradient Property Boundary

The nearest down-gradient property boundary location is near the SCR dam, located approximately 2.6 miles (13,728 feet) from Units 3 and 4. Performing an analysis similar to that used for the closest property boundary, a travel time of approximately 217.55 years is obtained for a release at the Glen Rose/Twin Mountain Formation contact below the CPNPP to reach the nearest potential down-gradient location of an unrestricted groundwater user.

Groundwater Volume within the Transport Pathway

Nearest Property Boundary

Based upon a 105.6 gallon release of source term contamination from the total 52,800 gallon release, which is assessed for the bounding case, a 5 ft x 5 ft contamination slug from the Glen Rose Formation is assumed to enter the Twin Mountain Formation (at either the CPNPP Unit 3 or 4 location) and travel to the nearest potential location of an unrestricted groundwater user (southwest property boundary, 3960 ft) where the groundwater volume within this pathway maintains this dimension along the pathway length. Maintaining this slug dimension along the Twin Mountain aquifer is considered conservative inasmuch as the source term would actually readily disperse, mix and dilute further in the aquifer. This volume is based on a Twin Mountain Formation average porosity of 27%. RCOL2_02.04.13-05, 06, 07 S01

- Cross sectional area = 5 ft x 5 ft = 25 ft^2
- Total volume of the transport pathway = 25 ft^2 x 3960 ft = 99,000 ft^3
- Groundwater volume (pore space) within the transport pathway = 99,000 ft^3 x 0.27 = 26,730 ft^3
- Converting to gallons = 26,730 ft3 x 7.48 gal/ft^3 = 199,940.4 gallons

Nearest Down-Gradient Property Boundary

An assumed 5 ft x 5 ft slug enters the Twin Mountain Formation at either the CPNPP Unit 3 or 4 location, and travels to the nearest potential down-gradient (east-southeast) location of an unrestricted groundwater user (SCR Dam, 13,728 ft) where the groundwater volume within this pathway maintains this dimension along the pathway length. This volume is based on a Twin Mountain Formation average porosity of 27%.

- Cross sectional area = 5 ft x 5 ft = 25 ft^2
- Total volume of the transport pathway = 25 ft^2 x 13,728 ft = 343,200 ft^3
- Groundwater volume (pore space) within the transport pathway = 343,200 ft^3 x 0.27 = 92,664 ft^3
- Converting to gallons = 92,664 ft^3 x 7.48 gal/ft^3 = 693,136.7 gallons

Concentration of BAT After Travel Time Through Glen Rose Formation Limestone

The radioisotopes from DCD Table 11.2-17 BAT concentrations, whose isotopic concentrations less than 1.0 x 10^{-3} in fraction of concentration limits are excluded from this evaluation are first decayed for 18.75 years, representing straight-line travel duration through the Glen Rose Formation limestone to the upper cap of the Twin Mountain Formation aquifer. Table 2.4.13-210 shows the resulting concentrations after 18.75 years of decay. As shown in Table 2.4.13-210, typical

isotopes such as tritium, cesium, strontium, cobalt and iron that readily disperse, mix, travel and are diluted by groundwater, are the few remaining isotopes to evaluate for the release. RCOL2_02.04.13-05, 06, 07 S01

Based upon the ratio of the high horizontal hydraulic conductivity of the existing fill materials (Path #1: 3.5 E-03 cm/sec; Path #2: 5 E-04 cm/sec; Subsection 2.4.12.3) to the low, conservative, assumed vertical hydraulic conductivity for the hypothetical case through the Glen Rose Formation (Path #3: 1 E-06 cm/sec), it can be conservatively concluded that only a fraction of the BAT contents (52,800 gallons) would be released vertically. The majority of the release will travel horizontally through the engineered fill that communicates with the existing fill and ultimately to SCR. The horizontal release discussed in Subsection 2.4.13.5 is assessed for 100% of the 52,800 gallons of BAT source term contamination since it is the most probable and plausible pathway with the fastest release time to a potential unrestricted potable water supply receptor. Since this hypothetical scenario assumes that advective groundwater transport vertically is possible through the Glen Rose Formation, the amount of groundwater flow vertically will be proportional to the percentage (ratio) of the vertical and hydraulic conductivities. Note that no credit is taken for any dispersion in the engineered fill as it has not yet been designed.

The percentage of the vertical to horizontal conductivities is calculated by:

$$\%K_v = (K_v / K_h) \times 100\% \qquad \text{Equation 4}$$

Where:

$\%K_v$ = vertical percentage of hydraulic conductivity

K_v = vertical hydraulic conductivity (ft/day)

K_h = horizontal hydraulic conductivity (ft/day)

The fraction of the total BAT released volume which travels vertically is then calculated by:

$$V_{BATv} = V_{BAT} \times \%K_v \qquad \text{Equation 5}$$

Where:

V_{BATv} = vertical volume of the BAT release, gal

V_{BAT} = total BAT release volume, gal

$\%K_v$ = vertical percentage of hydraulic conductivity

Determination of Path #1 Vertical Groundwater Volume

For the percentage of the vertical to horizontal hydraulic conductivities for a Path #1 BAT release, Equation 4 is modified as follows to determine $\%K_{v1}$: RCOL2_02.04.13-05, 06, 07 S01

$$\%K_{v1} = (K_v / K_{h1}) \times 100\% \qquad \text{Equation 6}$$

Where:

$\%K_{v1}$ = path #1 vertical percentage of hydraulic conductivity

K_v = vertical hydraulic conductivity (ft/day)

K_{h1} = path #1 horizontal hydraulic conductivity (ft/day)

$K_{h1} = 3.5\times10^{-3}$ cm/s = $((3.5\times10^{-3}$ cm/s) x (86,400 s/day))/30.48 cm/ft) = 9.921 ft/day

$K_v = 1.0\times10^{-6}$ cm/s = $((1.0\times10^{-6}$ cm/s) x (86,400 s/day))/30.48 cm/ft) = 2.83×10^{-3} ft/day

$\%K_{v1} = (K_v/K_{h1}) \times 100\% = (2.83\times10^{-3}$ ft/day)/(9.921 ft/day) x 100% = 0.03%

For the fraction of a Path #1 BAT released volume which travels vertically, Equation 5 is modified as follows to determine VBATv1:

$$V_{BATv1} = V_{BAT1} \times \%K_{v1}$$

Where:

V_{BATv1} = vertical volume of the BAT release for Path #1, gal

V_{BAT1} = total Unit 1 BAT release volume, gal

$\%K_{v1}$ = vertical percentage of hydraulic conductivity (from Equation 6)

V_{BAT1} = 52,800 gal

$\%K_{v1}$ = 0.03% = 0.0003

$V_{BATv1} = V_{BAT1} \times \%K_{v1}$ = 52,800 gal x 0.0003

V_{BATv1} = 15.1 gal

Determination of Path #2 Vertical Groundwater Volume

For the percentage of the vertical to horizontal hydraulic conductivities for a Path #2 BAT release, Equation 4 is modified as follows to determine $\%K_{v2}$: RCOL2_02.04.13-05, 06, 07 S01

$$\%K_{v2} = (K_v/K_{h2}) \times 100\%$$

Where:

$\%K_{v2}$ = path #2 vertical percentage of hydraulic conductivity

K_v = vertical hydraulic conductivity (ft/day)

K_{h2} = path #2 horizontal hydraulic conductivity (ft/day)

$K_{h2} = 5.0\times10^{-4}$ cm/s = ((5.0x10-4 cm/s) x (86,400 s/day))/30.48 cm/ft) = 1.42 ft/day

$K_v = 1.0\times10^{-6}$ cm/s = ((1.0x10-6 cm/s) x (86,400 s/day))/30.48 cm/ft) = 2.83×10^{-3} ft/day

$\%K_{v2} = (K_v/K_{h2}) \times 100\% = (2.83\times10^{-3}$ ft/day)/(1.42 ft/day) x 100% = 0.20%

For the fraction of a Path #2 BAT released volume which travels vertically, Equation 5 is modified as follows to determine VBATv2:

$$V_{BATv2} = V_{BAT2} \times \%K_{v2}$$

Where:

V_{BATv2} = vertical volume of the BAT release for Path #2, gal

V_{BAT2} = total Unit 2 BAT release volume, gal

$\%K_{v2}$ = vertical percentage of hydraulic conductivity

V_{BAT2} = 52,800 gal

$\%K_{v2}$ = 0.20% = 0.002

$V_{BATv2} = V_{BAT2} \times \%K_{v2}$= 52,800 gal x 0.002

V_{BATv2} = 105.6 gal

Conservatively, this hypothetical analysis utilizes the larger release volume from a Unit 4 vertical release through the Glen Rose Formation limestone to the upper

cap of the Twin Mountain Formation. Table 2.4.13-211 provides the following information:

RCOL2_02.04.13-05, 06, 07 S01

- Isotopes considered for the hypothetical release pathway from the 18.75 years of decay (Table 2.4.13-210) while traveling through the Glen Rose Formation limestone.
- Corresponding ECL for each isotope
- Resulting dilution effect from Twin Mountain Formation aquifer groundwater up-gradient to the CPNPP property boundary, which was previously discussed as the more conservative shorter duration travel time.
- Summation of the concentration of the 105.6 gallons of source term contamination after an additional 62.75 years of decay (while traveling up-gradient in groundwater in Twin Mountain Formation) to the ECL.

The sum of the source term concentration contamination to the ECL after 81.53 years of total travel time or decay time: Σ Isotope X (μCi/ml) / ECL X (μCi/ml) = 8.24

In this hypothetical scenario, Cs-137 is the source term isotopic contaminate which results in exceeding the ECL. However, and most importantly, the limit would not actually ever be exceeded for the following reasons:

- In order to calculate advective groundwater flow through the Glen Rose Formation limestone, two orders of magnitude higher (1 E-06 cm/sec) hydraulic conductivity was used to allow the source term to travel through the Glen Rose Formation. As discussed previously, packer tests (Table 2.5.4-206) in the power block areas show low hydraulic conductivities ranging from E-08 to E-09 cm/sec or no water takes) from plant grade elevation (822 ft msl) to 677 ft msl. Therefore, the site-specific packer tests showing low hydraulic conductivities support the conclusion in Subsection 2.4.13.4.1 that the vertical pathway is not plausible or probable.
- The horizontal pathways and the vertical pathways assume that the engineered fill surrounding the A/B is fully saturated, which readily provides a conducive pathway for the source term contaminated groundwater to readily travel through the engineered fill to the existing fill or the Glen Rose Formation limestone. It is realistic to expect that the engineered fill will not be fully saturated at all times. Thereby, allowing for retention, retardation and dispersion in the engineered fill. This supports the conclusions reached in Subsection 2.4.13.5 for the horizontal release Pathways #1 and #2. Additionally, the engineered fill will actually readily filter out the particulates in the source term leaving the remaining isotopes

that travel similarly to groundwater to mix and disperse with the surrounding groundwater.

RCOL2_02.04.13-05, 06, 07 S01

- The chemical characteristics of this source term are similar to the chemical composition of leaking tanks at other nuclear power plant decommissioned sites (Haddam Neck, Maine Yankee and Yankee Rowe, to name a few). Based upon extensive surveys and sampling analysis, the majority of the radioisotopes readily disperse near the genesis of the leak. The BAT source term chemical composition, like other leaking tanks at other sites, consists primarily of acidic tritiated water with some dissolved and un-dissolved metal particulates. Based upon this composition, this source term would readily disperse in the surrounding engineered fill with the metal particulate being retained in the surrounding concrete and engineered fill after the accident. The predominate radioisotopes that readily travel with groundwater are strontium, tritium and cesium, and this is confirmed by a review of the sampling and analysis results from the decommissioned sites final status survey results and groundwater monitoring program reports. Therefore, the source term would not exhibit slug travel behavior through the engineered fill or the Glen Rose Formation limestone, which is conservatively used in both the vertical and horizontal pathway evaluations.

Remediation and Restoration

Any NRC-licensed facility would be required under the terms and conditions of their license to immediately respond and begin remediation activities should an accident of this magnitude happen. It has been demonstrated that it would take approximately 81.53 years, using extremely conservative input parameters, to travel a straight-line pathway through the Glen Rose Formation to the Twin Mountain Formation aquifer and ultimately off the CPNPP property boundary. There is sufficient time (18.75 years through the Glen Rose Formation alone) to remediate and remove the source term prior to entering the upper cap of the Twin Mountains Formation. Therefore, this hypothetical accident would be fully remediated well before it enters the Twin Mountain Formation aquifer, and affect any potential unrestricted potable water supply user.

2.4.13.5 Liquid Effluent Groundwater Release Pathway to SCR and Summary Analysis Results

Potential groundwater pathways for the transport of contaminants to possible receptors are discussed in Subsection 2.4.12. These potential groundwater pathways are evaluated for a postulated release of the source term activity from the either CPNPP Unit 3 or 4 BAT in this subsection.

After evaluating alternative pathways, the most plausible pathway is groundwater transport of source term activity horizontally towards the east from CPNPP Unit 3, or towards the north from CPNPP Unit 4, to SCR surface water where the nearest receptor is located (Figure 2.4.12-212). The nearest receptor is considered to be

the Roto-cone gravity flow spillway device located at the south end of SCR (Figure 2.4.13-205). An existing Term Permit with the TCEQ, in accordance with the Brazos River Authority, CP-20 (Reference 2.4-296), Section 6.4.1, requires a minimum flow of 1.5 cfs be maintained at the Highway. 144 crossing over Squaw Creek, which eventually flows into the Brazos River. This requires a constant flow from the Roto-cone into Squaw Creek, which is verified at least daily by Luminant. Vertical migration of the source term from a postulated release is evaluated, but not considered a plausible pathway, for groundwater transport to the Twin Mountains Formation aquifer (Subsections 2.4.12.3 and 2.4.13.3). Groundwater transport west and south from either unit are also potential pathways (Subsections 2.4.12.3 and 2.4.13.2), but are not plausible based upon the hydrogeology and hydraulic gradients that exist pre-construction, and would exist post-construction.

The tank failure analysis focuses on the release of the entire 80% of the BAT source term from CPNPP Unit 3 because this pathway has the least amount of time through existing fill, least amount of SCR dilution and mixing volume, and the least amount of transport time to the Roto-cone. RCOL2_02.04.13-05, 06, 07 S01

As a result, the tank failure release analysis focuses on the bounding CPNPP Unit 3 pathway where the BAT source term activity could quickly be drawn into the CPNPP Units 1 and 2 circulating water (CW) intake (short-circuited) and be discharged closer to the release point, the Roto-cone device.

For the bounding CPNPP Unit 3 ~~p~~Pathway #1 (Figure 2.4.12-212), various cases of CW pump operation (no-flow, half-flow or full-flow) were considered to ensure the most bounding scenario is identified, and the resulting effect on mixing and dilution of the source term activity concentration (Table 2.4.13-203). RCOL2_02.04.13-05, 06, 07 S01

A postulated source term release from CPNPP Unit 4 Pathway #2, as depicted on Figure 2.4.12-212 is also considered a plausible groundwater pathway to enter SCR. The Unit 4 pathway is groundwater transport via existing fill where it will infiltrate into SCR. The source term activity transports via existing fill groundwater at a velocity of 1.01 ft/day (groundwater velocity) with an overall travel time of 346 days as compared to the Unit 3 pathway, where groundwater velocity is 4.13 ft/day for a travel time of 145 days (Table 2.4.12-211) over the 600 feet through existing fill to SCR (Figure 2.4.12-212). Slower travel time through existing fill with similar characteristics to CPNPP Unit 3 existing fill results in a greater dispersion of material, and larger water volume dilution effect. As depicted on Figure 2.4.12-214, once the source term activity infiltrates at the groundwater interface, it will slowly diffuse into SCR surface water. As the source term activity diffuses further into SCR surface water, it will be transported southward with surface water flow. As depicted on Figures 2.4.12-212 and 2.4.13-206, the influence of the CPNPP Units 1 and 2 CW pumps affects surface water flow, especially during summer months with very little inflow into SCR. The source term activity would most likely become entrained in the CW intake and exit similarly to the CPNPP Unit 3 release. Thus, a larger volume of SCR could be credited for this release. RCOL2_02.04.13-05, 06, 07 S01

Because the ECLs are met for the CPNPP Unit 3 cases of no-flow, full-flow or half-flow of CW pump (Subsection 2.4.13.5.4 through 2.4.13.5.6), the ECLs are also met for the Unit 4 diffusion case since additional diffusion time and SCR surface water volume could be credited.

This tank failure analysis concludes that, using the most conservative analysis, the BAT activity concentration will be sufficiently diluted by a portion of the existing fill groundwater and further diluted and mixed with SCR water to meet the ECLs specified in 10 CFR 20, Appendix B, Table 2.

The following factors or calculations are utilized in assessing the source term activity concentrations from a postulated release from either CPNPP Units 3 or 4 to the nearest plausible receptor (Roto-cone):

- The source term activity for the BAT was calculated using the RATAF code with 1 percent fuel defect, scaled down to 0.12 percent fuel failure, with appropriate tank factors applied.
- The calculated source term activity concentration remaining after 0.4 years or 145 days of decay is provided in Table 2.4.13-202.
- Potential groundwater pathways are CPNPP Unit 3 to the east or CPNPP Unit 4 to the north (Figure 2.4.12-212).
- Groundwater velocity travel time (Table 2.4.12-211).
- Volume of groundwater available for source term activity dilution.
- Volume of SCR surface water available for source term activity dilution.
- Mixing rate in SCR based upon half-flow or full-flow CW pumps.
- Diffusion in SCR with no-flow CW pumps operating.

In developing the most conservative scenarios, the following are not factored into the analysis. If factored into the analysis, these would provide much lower concentrations at the receptor: RCOL2_02.04.13-05, 06, 07 S01

- The source term chemical composition is acidic tritiated water with both dissolved and un-dissolved metal particulates. Based upon this composition, this source term would readily disperse and be retained in the surrounding engineered fill with the metal particulate being retained in the surrounding concrete and engineered fill. Therefore, the assumption regarding slug transport migration through both the engineered and existing fill is very conservative. RCOL2_02.04.13-05, 06, 07 S01
- No credit is taken for travel time through the engineered fill into the overall groundwater transport time. This is conservative because travel time

increases and allows for additional decay time, dilution, retardation and retention, thereby further reducing the source term activity concentration prior to reaching SCR.

- No credit is taken for retardation, retention or dilution in the engineered fill. This is conservative as these effects would further reduce the source term activity concentration.

- The engineered fill surrounding the ESW tunnel in communication with the existing fill on the east side of the ESW tunnel as depicted on Figures 2.4.12-213 (CPNPP Unit 3 pathway) and 2.4.12-214 (CPNPP Unit 4 pathway) is completely saturated. This is conservative because it allows for the source term activity as a slug to be transported to the existing fill where it subsequently infiltrates into SCR. The engineered fill will not likely be in complete communication with the existing fill and it will not likely be completely saturated at all times allowing for retention, retardation and dilution.

- For the case of both Units 1 and 2 operating, or either unit operating, the only SCR water credited for dilution or mixing is the Units 1 and 2 CW pump volume of either 2 million or 1 million gallons per minute, respectively. This is extremely conservative because a considerably larger volume of SCR water could be credited for dilution purposes (Figures 2.4.13-205 and 2.4.13-206 and Subsection 2.4.13.5.6). As shown on Figures 2.4.13-205 and 2.4.13-206, water dilution from the CW discharge to the Roto-Cone device would further dilute and mix the source term concentration prior to exiting via the Roto-Cone device. RCOL2_02.04.13-05, 06, 07 S01

- Only a portion (25 percent) of the total available groundwater is assumed to be available for dilution. This is conservative because a considerable amount of groundwater (approximately 9.98E06 gal) can be found in the existing fill that communicates with SCR. As discussed previously, the chemical composition of the source term contamination is acidic tritiated water with dissolved and un-dissolved metal particulates, and will travel readily with the groundwater first in the engineered fill and then through the existing fill. The source term would readily mix, disperse, and migrate with groundwater in the existing fill. RCOL2_02.04.13-05, 06, 07 S01

Monitoring wells in the existing fill indicate steady groundwater quantities with changes attributed to elevation rise and fall in SCR or rainwater infiltration. The hydraulic gradient in the existing fill remains steady as discussed in Subsection 2.4.12.3. Based upon the steady state groundwater quantity in the existing fill, it is reasonable and conservative to use only a portion of groundwater in the existing fill to dilute the source term contamination, especially considering that no credit is taken for the dilution, retention and retardation effect of the engineered fill. Further, it is assumed that the source term is traveling as a slug through the existing fill when in reality it would readily disperse and be retained first in the engineered fill and then in the existing fill. Further, there is no motive force to allow

for the infiltration of the existing fill groundwater into SCR except when there is an elevation change in SCR. Therefore, using a portion of a steady-state quantity of the existing fill groundwater is considered conservative when a much larger quantity of the groundwater could be credited (Subsections 2.4.12.3 and 2.4.13.5.4). Smaller percentages of the existing fill groundwater could be credited in the calculation. However, based upon the chemical composition of the source term contamination, it is reasonable to expect the contamination to readily disperse, dilute, and migrate with the groundwater. Further, even if no existing fill groundwater is assumed to dilute the source term, there is a considerably larger quantity of SCR water that could be credited at the CW pumps inlet and discharge toward the Roto-Cone device (estimated 3.66E09 gallons; Subsection 2.4.13.5.6) that would reduce the source term contamination below the ECLs prior to exiting the Roto-Cone device. SCR water volume credited for dilution was the volume at the CW pumps (2E06 gallons) with both units in operation to provide the greatest driving force to the Roto-Cone device. Two million gallons at the CW pump intake represents only 0.055% out of total available 3.66E09 gallons. Therefore, even if no credit is taken for existing fill groundwater dilution, there still exists significant available water quantities in SCR (not credited in the calculation) at the inlet and discharge to the CW pumps that would reduce the source term contamination below the ECLs prior to exiting the Roto-Cone device. RCOL2_02.04.13-05, 06, 07 S01

The following subsection describes the bounding CPNPP Unit 3 pathway scenario to the nearest receptor (Roto-cone gravity drain device).

2.4.13.5.1 Bounding CPNPP Unit 3 Pathway Scenario

A postulated release from CPNPP Unit 3 is the most conservative scenario. It is assumed that a physical phenomenon occurs causing the BAT to rupture and its contents spill to the floor or sides of the A/B (El. 785 ft, which is adjacent to the engineered fill outside the A/B). The tank is assumed to be 80 percent full in accordance with BTP 11-6. The bottom of the BAT cubicle is at El. 793 ft. As shown on Figure 2.4.13-201, the engineered fill is just outside of the BAT cubicle area in the A/B and around the R/B. Since the engineered fill has not been specified at this time, it is also assumed that the source term moves as a slug volume through the groundwater in the fully saturated engineered fill. This is very conservative because it is highly unlikely that the engineered fill would be fully saturated throughout the travel pathway. Additionally, travel through the saturated engineered fill increases travel time, and allows for dispersion and retardation that is not credited in the analysis.

The engineered fill surrounding the ESW tunnel is in contact with the existing fill on the east side of the ESW tunnel as depicted on Figure 2.4.12-213. As depicted on Figures 2.4.12-213 and 2.4.13-201, a stormwater retention pond is located east of Unit 3 that has an overflow elevation of approximately 810 ft msl,and a bottom elevation of approximately 800 ft msl. Groundwater elevations within the existing fill will be approximately equal to the surface elevation of SCR. For the purpose of the existing fill groundwater calculation, an SCR minimum operating elevation of 770 ft msl was used. The bottom of the stormwater retention pond is

located within the existing fill east of CPNPP Unit 3, and is approximately 30 feet above the groundwater surface within the existing fill. Therefore, the presence of the stormwater retention pond will not affect the existing fill groundwater volume, nor intercept groundwater impacted by the postulated release from CPNPP Unit 3. Although not expected, recharge from the stormwater retention pond would serve to produce a shallower groundwater gradient, thereby producing a slower groundwater velocity and travel time for the postulated release and a less conservative analysis of groundwater transport from CPNPP Unit 3. The existing fill is in communication with the SCR surface water.

Based upon site-specific hydrogeological data, the groundwater travel time through the existing fill is 145 days. Groundwater velocity within the existing fill material is based on (Table 2.4.12-211):

- The engineered fill surrounding the ESW pipe tunnel is saturated to a maximum groundwater elevation of Elevation High (E_h) = 820 ft msl.
- SCR operating low range is used for volume calculations (before makeup from Lake Granbury) elevation (E_l) = 770 ft msl.
- Distance to SCR (L_G) from the ESW and groundwater interface = 600 ft.
- Groundwater hydraulic gradient $(E_h-E_l) / L_G$ = 0.0833 ft/ft.
- Hydraulic Conductivity (K_h) of the existing fill material = 3.50E-03 cm/sec = 1.15E-04 ft/sec = 9.92 ft/day.
- Effective Porosity (η_e) = 0.2.
- Velocity (V) of groundwater through existing fill = $(K_h (E_h-E_l) / L_G) / \eta_e$ = 4.13 ft/day.
- Groundwater travel time (T) $T = L_G / V$ = 0.4 years or 145 days.

Table 2.4.13-202 shows the source term activity concentration remaining after 145 days of decay from the initial activity concentrations in DCD Table 11.2-17. As shown in Table 2.4.13-202, some of the isotopes are at or below the ECLs. Therefore, any dilution will reduce these concentrations well below the ECLs. From Table 2.4.13-202, the primary radioisotopes of consideration are H-3, Fe-55, Co-58, Co-60, Sr-90, Cs-134, and Cs-137, which are typically the primary radioisotopes contributing to groundwater contamination.

2.4.13.5.2 Modeling Equations Used in the Tank Failure Analysis

Figure 2.4.13-202 diagram depicts the simple process equations used in modeling the source term activity flow, dilution effects and mixing once the source term

activity infiltrates into SCR from the groundwater. The governing differential equations for the time-dependent activity in each compartment are the following:

$$\frac{dA_A}{dt} = S_A + F_{CW,B}[A_B(t)] - (\lambda + F_{env,A} + [F_{CW,A} - F_{env,A}])[A_A(t)] \quad \text{Eq. 1}$$

$$\frac{dA_B}{dt} = -(\lambda + F_{CW,B})[A_B(t)] + ([F_{CW,A} - F_{env,A}])[A_A(t)] \quad \text{Eq. 2}$$

Where:

$F_{CW,i}$ = Normalized circulation water flow for Units 1 and 2 for compartment "i" [1/hr], defined as $F_{CW,i} = F_{CW}/V_i$

$F_{CW,i}$ = Circulation water flow for Units 1 and 2 [gallon/h]

$F_{env,\,i}$ = Normalized flow to the environment for compartment "i" [i/hr], defined as $F_{env,i} = F_{env}/V_i$

F_{env} = Flow to the environment [1/hr],

λ = Decay coefficient [1/hr],

S_A = Constant source for compartment A [μCi/hr], and

A_i = Activity in compartment "i" [μCi].

The following assumptions are included in this model:

- The source term activity infiltration rate into SCR is assumed to be constant.
- The flow to the environment is negligible (conservative for concentration calculations because it retains all of the activity in SCR).
- Only long-lived isotopes are considered; therefore, radioactive decay is neglected prior to the source term being completely infiltrated into SCR.
- SCR is at constant level (no significant changes in volume due to rainwater or other water sources being added provides conservatism because it retains the activity in SCR).
- Following the release of all the source term, the concentration decreases with time due to mixing with the large SCR bulk volume available for dilution (1.73E10 based upon the CW discharge volume plus the recirculation volume in SCR).

Using these assumptions, the equations simplify to:

$$\frac{dA_A}{dt} = S_A - (F_{CW,A})[A_A(t)] \quad \text{Eq. 3}$$

$$\frac{dA_B}{dt} = F_{CW,A}[A_A(t)] \quad \text{Eq. 4}$$

The SCR mixing volume (Volume "SCR_A") while the source is being added becomes:

$$A_A(t) = A_A(t = 0)e^{-(F_{CW,A})t} + S_A\left[\frac{1 - e^{-(F_{CW,A})t}}{F_{CW,A}}\right] \quad \text{Eq. 5}$$

Because the activity is deposited in the SCR bulk volume, the source is assumed to be constantly added to the volume over the release period. No activity from the tank is assumed to be present in SCR prior to the event; therefore, the final equation during the release phase becomes:

$$A_A = S_A\left[\frac{1 - e^{-(F_{CW,A})t}}{F_{CW,A}}\right] \quad \text{Eq. 6}$$

Based on the above simplified equation, as time progresses, the equilibrium concentration simplifies to:

$$A_{A,eq} = \frac{S_A}{F_{CW,A}} \quad \text{Eq. 7}$$

Because:

$$\lim_{t \to \infty}(1 - e^{-(F_{CW,A})t}) = 1 \quad \text{Eq. 8}$$

Therefore, to calculate the maximum concentration this model Equation is used. Note that this conservatively assumes that equilibrium is achieved prior to the source being depleted.

The equilibrium concentration in compartment A can then easily be determined by:

$$C_{A,eq} = \frac{A_{A,eq}}{V_A} = \left(\frac{1}{V_A}\right)\left(\frac{S_A}{F_{CW,A}}\right) = \left(\frac{S_A}{(V_A)\left(\frac{F_{CW}}{V_A}\right)}\right) = \frac{S_A}{F_{CW}} \quad \text{Eq. 9}$$

2.4.13.5.3 Infiltration Area of Existing Fill Groundwater and Effect on Volumetric Flow Rate into SCR

As shown on Figure 2.4.12-213, the source term travel through the existing fill initially is at a higher elevation than the existing fill adjacent to the SCR embankment, thus producing the hydraulic gradient (Subsection 2.4.12.3). Once the source term reaches the groundwater in the existing fill that forms the embankment and infiltration point to SCR, the only motive force of infiltration to SCR is a change in SCR elevation. RCOL2_02.04.13-05, 06, 07 S01

Due to the ~~hydrostatic~~ pressure head of SCR pushing against the existing fill surface area (Figure 2.4.12-213), where the groundwater in the existing fill communicates with SCR, it is realistically expected that the groundwater infiltration rate is much, much slower. ~~Groundwater~~The motive force for groundwater infiltration into SCR from existing fill ~~would most likely occur at times when SCR hydrostatic pressure is decreasing due to a change in level or a considerable temperature change~~is due to elevation changes in SCR. However, to determine the actual flow infiltration to SCR would require another model and more data acquisition. As a result, the flow into SCR from the existing fill is assumed to occur at the groundwater volumetric flow rate through the existing fill. This is conservative because the groundwater flow rate through the existing fill does not have enough ~~driving~~motive force to infiltrate at this rate ~~when compared to the hydrostatic head of SCR~~with little fluctuation in SCR elevation. A discussion on the effect of a smaller infiltration surface area and its effect on infiltration rate and dilution in SCR follows. RCOL2_02.04.13-05, 06, 07 S01 RCOL2_02.04.13-05, 06, 07 S01

The existing fill material is an irregular surface. However, the cross sections (Figure 2.4.13-203 and 2.4.13-204) reveal that it is roughly equivalent to one-half of a reposed conical shape with an elliptical base. Therefore, the fill volume below 770 ft msl was conservatively calculated as one-half the volume of an elliptical-based cone with basal surface area twice that of the calculated infiltration area from cross section 3c and a length equivalent to the distance of the farthest existing fill base at 770 ft msl (Figure 2.4.13-203). This results in a total fill volume below 770 ft msl of 6,671,033.8 cu. ft. and a total infiltration surface area of 34,854.49 sq. ft. Elevation 770 ft msl is conservatively chosen as SCR surface water level, which is the lower end of the normal SCR operating range, and provides the least amount of dilution volume and ~~hydrostatic~~ pressure head for the analysis. RCOL2_02.04.13-05, 06, 07 S01

Multiplying the total fill volume and infiltration area by the effective porosity of 0.2 yields a groundwater volume of approximately 9.98 million gallons and an effective infiltration surface area of approximately 6970.9 ft^2. This is also a conservative assumption because the slug of source term activity would have to have ~~dispersed across this entire area for this to occur~~readily dispersed across the entire area since the chemical characteristics of the source term are tritiated acidic water with metallic particulates. The infiltration flow rate of groundwater into SCR is given by: RCOL2_02.04.13-05, 06, 07 S01

F_{GW} – flow rate of contaminated groundwater to SCR

A_{GW} – Area of existing fill groundwater contribution

V_{GW} – Velocity of groundwater in existing fill

$F_{GW} = A_{GW} * V_{GW}$ = 6970.9 ft_2 * 4.13 ft/day = 28,789.8 ft3/day or 149.7 gpm

Using the volumetric flow rate of 149.7 gpm as the infiltration rate into SCR is extremely conservative inasmuch as this was based upon the entire half-elliptical cone surface infiltration area of 6970.9 ft^2, which would have required the source term activity to disperse and dilute throughout the existing fill for this to occur. Using this volumetric flow rate is also conservative because ~~the SCR hydrostatic head is much greater resulting in very little actual infiltration into SCR~~the only motive force for the existing fill groundwater infiltration into SCR is SCR elevation changes. RCOL2_02.04.13-05, 06, 07 S01

The source term activity, however, is assumed to move as a slug through the existing fill where it would not readily disperse over the entire surface area of the half elliptical cone base. If only the effective surface area of the BAT is considered as infiltration area, the resulting infiltration rate is much slower and longer time to flow into SCR.

~~The surface area for t~~The BAT ~~is based upon~~ DCD general arrangement drawing is shown in Figure 1.2-29 ~~that shows a BAT diameter of approximately 19 feet~~. CTS-01351 Actual dimensions of the BAT have not been designated; however, using an approximate 19 foot diameter tank top or bottom is a close approximation of actual dimensions of the top or bottom of the BAT. Thus, the surface area is $\pi d^2/4$ = 283.5 ft^2, and can be used to demonstrate the slug surface area form traveling in the existing fill groundwater from the engineered fill. It should be noted that this is an extremely conservative assumption due to the chemical characteristic of the source term being primarily acidic tritiated water with metallic particulate and would readily disperse into the existing groundwater. However, using the slug surface area of the BAT as an approximate slug form, the groundwater flow rate of the slug source term into SCR can be estimated from the following: RCOL2_02.04.13-05, 06, 07 S01

$F_{GW} = A_{GW} * V_{GW}$ = 283.5 ft^2 * 4.13 ft/day = 1179.1 ft^3/day or 6.12 gpm

The source term slug flow rate into SCR is 24 times slower than the half-elliptical cone infiltration rate of 149.7 gpm where the source term is dispersed across the entire existing fill surface area. In either case, actual infiltration rate of groundwater into SCR is much slower and predicated upon SCR elevation changes occurring. Since SCR elevation levels are fairly constant, actual infiltration rates of contaminated groundwater from the source term are extremely slow. RCOL2_02.04.13-05, 06, 07 S01

This demonstrates that actual infiltration into SCR is slow and would allow for considerable mixing of SCR water with a portion of the source term becoming entrained in the recirculating water flow back to the CW intake pumps. Thus, actually providing a much greater dilution volume.~~with the time it takes a smaller surface area of source term activity mixed with the groundwater to flow into SCR, a portion of the activity will combine with the recirculating water flow back to the intake through SCR, providing a much greater dilution volume.~~ It also demonstrates that choosing a high volumetric flow rate as the infiltration rate into SCR is very conservative because this infiltration rate would be indicative of the source term activity dispersing, mixing and diluting with the entire half elliptical cone surface area groundwater. Finally, using the higher infiltration rate of 149.7 gpm is very conservative considering that the actual infiltration rate into SCR is much, much slower due to ~~the hydrostatic head difference between SCR and the existing fill~~small SCR elevation changes. RCOL2_02.04.13-05, 06, 07 S01

RCOL2_02.04.13-05, 06, 07 S01

2.4.13.5.4 Dilution Effect of the Existing Fill Groundwater

Because a dispersion model with additional groundwater and soil data would need to be taken to predict the dilution, retardation and retention effects of the existing fill groundwater, only 25 percent of the total amount available is conservatively credited in the tank failure analysis. ~~It is reasonable to credit 25 percent of the existing fill groundwater because the source term activity has been conservatively assumed to be moving as a slug through the engineered fill before it reaches the existing fill with no credit taken for dilution, retardation, retention or dispersion. Once the source term activity reaches the existing fill, it will disperse, mix with and be diluted by some of the existing fill groundwater.~~ As discussed in Subsection 2.4.13.5.3, ~~due to the hydrostatic head difference between the existing fill and SCR, there is a considerably longer stay time in the existing fill groundwater before it would infiltrate into SCR, thus allowing for greater dilution, retardation and dispersion of source term activity.~~the only real motive force for infiltration into SCR occurs when there is a change in SCR elevation or precipitation, resulting in greater dilution, retardation and dispersion of the source term activity with the existing fill groundwater. The dilution effect of crediting various quantities of existing fill groundwater is provided in Table 2.4.13-204. RCOL2_02.04.13-05, 06, 07 S01

Using the concentration of each radioisotope from the effects of just 25 percent dilution from the existing fill groundwater gives the source term activity concentration into SCR for the conservatively larger infiltration area rate of 149.7 gpm (Table 2.4.13-205).

When it is realistically assumed that some (25 percent) groundwater dilution, retardation and retention occurs, the total activity takes 16666.67 min (277.78 hrs or 11.6 days) to infiltrate into SCR. This conservative infiltration rate for groundwater infiltration over the one-half elliptical shape shows that the infiltration is not instantaneous, that there is some expected retardation and retention by the existing fill groundwater, and that over the 11.6 days to completely infiltrate into SCR, a portion of the activity would be combined with the recirculation flow back

to the CW intake; thus, a larger SCR water volume could be credited for the recirculation flow (Figure 2.4.13-206).

2.4.13.5.5 Effects of Circulating Water Pump Operation on Mixing and Dilution

Based upon the simplified Equation 9 in Subsection 2.4.13.5.2, the small dilution effect of Units 1 and 2 CW pumps at maximum capacity (2.0E06 gpm) or one Unit's CW pumps operating at maximum capacity (1.0E06 gpm) reduces the source term activity below the ECLs (Table 2.4.13-206).

The 25 percent dilution effect of the total available existing fill groundwater, with the higher infiltration rate into SCR (149.7 gpm), mixing with the CW intake at 2.0E06 gpm or 1.0E06 gpm, demonstrates that the ECLs are met. The Summation ($\sum$) of the total activity concentration as a ratio of the ECL < 1.0 is shown in Table 2.4.13-207 for the 149.7 gpm infiltration flow rate into SCR from existing fill groundwater for maximum CW pump operation (2.0E06 gpm).

Where:

$\sum$ (Concentration Nuclide / ECL Nuclide) < 1.0

$\sum$ (Concentration Nuclide / ECL Nuclide) = 3.2E-01 at the 149.7 gpm infiltration rate is well below 1.0 for all CW pumps operating at 2.0E06 gpm.

For the case of half-flow CW pumps operating at maximum capacity (1.0E06 gpm), the ratio of activity concentration to the ECL is provided in Table 2.4.13-208.

$\sum$ (Concentration Nuclide / ECL Nuclide) = 6.43E-01 at the 149.7 gpm infiltration rate is well below 1.0 for half the CW pumps operating at 1.0E06 gpm.

2.4.13.5.6 Dilution Effect and Mixing of SCR

Once the source term activity infiltrates into SCR through the existing fill (calculated to be approximately 145 days), the source term activity will enter SCR and be drawn into the CPNPP Units 1 and 2 CW intake pumps and discharged to the south side of the CPNPP Unit 1 and 2 peninsula at 2 million gpm, where it will eventually encounter the Roto-cone drain to SCR spillway. Because the Roto-cone gravity flow device constantly discharges water to Squaw Creek and ultimately to the Brazos River in order to meet the TCEQ Term Permit CP-20 described previously, the limiting case for dilution becomes when both CPNPP Units 1 and 2 are in operation and the CW pumps are running at 2 million gpm (greatest driving force with least amount of time to reach the Roto-cone gravity flow device). Therefore, the CW discharge point becomes the location for highest source term concentration prior to dilution by SCR discharge volume. The entire 11.6 days release duration is irrelevant because some source term activity would combine with recirculating water back to the CW intake (greater dilution volume) and some activity could potentially reach the Roto-cone and be released to the

environment during the first minute. Both CW pumps fully operating provides the greatest driving force and sufficient mixing for the contamination to reach the Roto-cone in the shortest time.

The flow from the CW pumps will potentially reach the Roto-cone fairly rapidly and only be diluted (11,217 ac-ft. or 3.66E09 gallons) by the effect of the small CW intake volume plus the discharge CW volume on the opposite side of the peninsula (Figure 2.4.13-205). The CPNPP Units 1 and 2 CW pumps provide a strong driving and mixing force for the dilution of the source term activity. No-flow conditions are also examined due to the possibility of CPNPP Units 1 and 2 eventually being decommissioned during the life of CPNPP Units 3 and 4, or both CPNPP Units 1 and 2 in an outage. As shown on Figure 2.4.13-206, no water volume in the inlet areas, intake area or the discharge area is included. A detailed flow model of SCR has not been performed. Thus, only an estimate of this water volume can be attributed to recirculation flow from CW discharge to CW intake.

SCR volume was calculated using bathymetry data from a July 11, 2007 bathymetry study (Reference 2.4-297). If the CW pumps were not operating at full capacity or one unit was down, there would be a lower driving force to reach the Roto-cone, and a greater volume of water to dilute the source term activity due to the recirculating water volume east of the existing fill area of SCR plus the water volume north of the Roto-cone plus the discharge point on the south side of the peninsula. This would result in dilution of the source term concentration well below the ECLs prior to discharge at the Roto-cone (Figures 2.4.13-205 and 2.4.13-206).

The mixing volume for half-flow operations is the mixing volume shown on Figure 2.4.13-207, Area 1 (11,217 ac-ft. or 3.66E09 gallons) plus the mixing volume from Area 3 (41,757 ac-ft. or 1.36E10 gallons) for a total of 1.73E10 gallons. This volume does not include depths in SCR greater than 66 feet. This is a conservative assumption because some mixing would most likely occur at greater depths in SCR, depending on the CPNPP Units 1 and 2 operating conditions, depth in SCR, seasonal fluctuations, rain events or other conditions that effect temperature changes in SCR. As a result, no credit is taken for water dilution at El. 704 ft. or deeper. The volume does not include any contribution from inlets or areas where it is expected that CW discharge would not have a credible effect on diffused dilution or mixing. Recirculation flow time to the intake is unknown and depends on CW flow rate, SCR level, time of year, where in the fuel cycle the unit is operating, and other parameters. However, using the CW pumps in full operation provides the greatest driving force and allows for a simple estimate of the recirculation time:

1.73E10 gal / 2E06 gpm = 8635 min or 143.92 hours or 6 days recirculation time

The time for complete source term activity infiltration into SCR from existing fill is 11.6 days, which is greater than the recirculation flow time. Therefore, additional SCR dilution volume from CW recirculation flow (Figure 2.4.13-206) can be credited.

For no-flow conditions (Figure 2.4.13-206), the source term activity would diffuse with the water volume east of the existing fill and very slowly diffuse southward toward the Roto-cone release point because the Roto-cone discharge rate to Squaw Creek would be the only driving force in this scenario. Using the bathymetry study described previously, an estimated volume of SCR water at no-flow conditions is 41,757 ac-ft. or 1.36E10 gallons. This volume does not include inlet areas close to the existing fill release point, nor does it include depths greater than 66 ft. in SCR where it is not expected that much mixing or diffusion will occur. Additionally, it is unknown how long it would take the diffused source term water volume to flow southward towards the Roto-cone release point.

No-flow conditions would result in the source term activity infiltrating SCR via the existing fill groundwater interface and slowly diffusing into the SCR water adjacent to the east side of CPNPP Unit 3. As shown on Figure 2.4.13-206, no water volume in the inlet areas or intake area is included. The credited volume as discussed previously is 1.36E10 gallons and does not include any water below a depth of 66 feet in the reservoir. The infiltration rate into SCR is discussed previously, but in this case is irrelevant as diffusion throughout SCR surface water would be very slow. The only driving force to reach the Roto-cone area is the discharge through the Roto-cone. An additional model would have to be developed to calculate the diffusion rate of source term activity into SCR and the time to reach the Roto-cone. However, Table 2.4.13-209 shows that the ECLs would be met before any contamination reached the Roto-cone by simple diffusion with the SCR surface water above the 66 ft depth. In this case, no credit is taken for dilution effect of existing fill groundwater. If credit were taken, the resulting ratio of activity to ECL would be further diminished as demonstrated in Subsection 2.4.13.5.5.

$\sum$ (Concentration Nuclide / ECL Nuclide) = 7.87E-01

2.4.13.5.7 Summary

Considerable conservative assumptions include:

- No credit taken for the dilution, retardation or retention effects of the engineered fill;
- No credit taken for the travel time through the engineered fill that is assumed to be completely saturated;
- The source term activity moves as a slug volume through both the engineered fill and existing fill;
- The infiltration rate into SCR is one-half elliptical cone surface area of the existing fill (149.7 gpm). This flow rate is excessive when compared to actual very slow infiltration into SCR resulting from a decrease in ~~hydrostatic head~~elevation between SCR and the adjacent existing fill surface area in communication with SCR; RCOL2_02.04.13-05, 06, 07 S01

- ~~Crediting only 25 percent of the existing fill groundwater when actually there would be greater dispersion, dilution and retention in the groundwater.~~Although a steep gradient within the existing fill volumes was used to provide a conservative groundwater velocity for the accidental release analysis (fastest travel time to SCR), in reality the groundwater elevation within the existing fill is relatively flat and in equilibrium with the surface water elevation of SCR. Changes in SCR elevation is rapidly communicated through the groundwater in the isolated existing fill material. The elevation of SCR surface normally varies by a few tenths of a foot (Section 2.4.12.3). The water elevation is fairly constant with action required if the level drops below 770 ft msl. Normal variations in SCR elevation would result in small recharge and discharge events, dependant on SCR operations and rainfall events. RCOL2_02.04.13-05, 06, 07 S01

  These multiple small recharge and discharge events, as well as rainfall infiltration, would promote lateral and vertical mixing of contaminates entering the existing fill from the engineered fill. Although expected to mix readily throughout the existing fill due to the slow net movement of groundwater towards SCR, only 25% of the groundwater volume within the existing fill is used to provide a conservative estimate for dilution.

- Using the surface area of the one-half elliptical cone existing fill volume demonstrates that there would have to be greater dispersion in the groundwater; and

- For the limiting case, crediting only the 2 million or 1 million gpm mixing and dilution flow of CW intake when further dilution will occur based upon the CW discharge volume prior to reaching the Roto-cone release point.

Additionally, it has been adequately demonstrated that a smaller infiltration flow rate from the existing fill into SCR results in a longer time for the total activity to infiltrate into SCR. This longer infiltration time (11.6 days) ensures a larger dilution volume because some of the source term activity will combine with recirculation flow and be diluted by the bulk volume of SCR. Furthermore, it has been demonstrated that adequate mixing occurs in SCR using the mixing driving force of the CW pumps only. For no-flow pump conditions, it is demonstrated that simple diffusion and dilution by SCR surface water is adequate to meet the ECLs for the case of either a Unit 3 or 4 tank failure without crediting existing fill groundwater dilution.

Crediting 25 percent of the existing fill groundwater for dilution of the source term activity prior to entering SCR, combined with the slow infiltration effect of the existing fill groundwater into SCR, and only the mixing and dilution effect of the CW intake of either 1 or 2 million gpm results in meeting the ECLs for all radioisotopes that infiltrate into SCR via the existing fill groundwater. The unrestricted potable water supply receptor location is the Roto-cone discharge area in the southeast portion of SCR near the Squaw Creek dam. All activity concentrations reaching the Roto-cone device have been shown to be below the

limits of 10 CFR 20, Appendix B, Table 2, and thus the requirements of 10 CFR 20.1301, 20.1302 and 10 CFR 100 are satisfied.

2.4-298 C.-Y.Wang, M. Manga, Earthquakes and Water, Lecture Notes in Earth Sciences 114, 218p, Springer Verlag Berlin Heidelberg, 2010. RCOL2_2.4.12-09 S01

2.4-299 U.S. Geological Survey, Ground Water Atlas of the United States, Oklahoma, Texas, HA-730-E, Website, http://pubs.usgs.gov/ha/ha730/ch_e/E-text8.html, Accessed, July 19, 2011, Table 7. RCOL2_02.04.13-05, 06, 07 S01

Table 2.4.12-211
Groundwater Velocities and Travel Times

RCOL2_02.04.12-8 S01

	Path 1	Path 2	Path 3
Release Elevation (E_{hH})(ft msl)	820.00	820.00	820.00
Discharge Elevation (E_{hL})(ft msl)	770.00	770.00	592.00
Distance to SCR (L)(ft)	600	350	193
Hydraulic Gradient (Eh_H-El_L)/L	0.0833	0.1429	1.18
Velocity (V) (ft/day)	4.13	1.01	0.028
Travel Time (T) (days)	145	346	6858
(years)	0.40	0.95	18.78

Path 1 is from Unit 3 east to SCR; Path 2 is from Unit 4 north to SCR; Path 3 is from the top of the A/B leveling concrete to the Twin Mountains Formation.

Equation for Velocity: ~~V~~V = (K_h (E_{hH}-E_{hL})/L)/η Equation for Travel Time: T = L/V

Path 1 fill K_{hH} is 3.50 x 10^{-3} cm/sec (9.92 ft/day) from RW-1 recovery test.

Path 2 fill K_{hH} is 5.00 x 10^{-4} cm/sec (1.42 ft/day) from MW-1219a slug test.

Path 3 K_V is assumed to be 1.00 x 10^{-6} cm/sec (2.83 x 10^{-3} ft/day).

Conversions: 1 day = 86,400 seconds; 1 foot = 30.48 centimeters; 1 year = 365.25 days.

Assumptions:

1. Engineered fill is conservatively assumed as having negligible transport time and fully saturated to 820 ft msl.
2. ~~Engineered fill is assumed to be fully saturated to level of the perimeter trench drains.~~Maximum Elevation (E_H)
   a. Paths 1 and 2: the maximum groundwater elevation is assumed to be 820 ft msl, above the elevation of the trench drain, transposed to the edge of the existing fill at the pathway release point (E_H @ 820 ft msl).
   b. Path 3: the maximum groundwater elevation is assumed to be 820 ft msl, above the elevation of trench drain (E_H @ 820 ft msl).
3. ~~Release elevation is assumed to be the elevation of trench drain transposed to the edge of the existing fill at the pathway release point (E_h at 820 ft msl).~~

Discharge Elevation (E_L)

   4a. Paths 1 and 2: the ~~D~~discharge elevation is assumed to be the elevation of the SCR minimum operating pool (E_{hL} at 770 ft msl).
   b. Path 3: the discharge elevation is assumed to be the elevation of the Glen Rose/Twin Mountains Formation contact beneath Units 3 and 4 (E_L @ 592 ft msl).

4. Pathway Distance (L)
   ~~5~~a. Paths 1 and 2: the ~~P~~pathway distance is assumed to be the shortest distance from the pathway release point (nearest engineered/existing fill interface) to the shoreline of SCR.
   b. Path 3: the pathway distance is assumed to be the vertical distance from top of the A/B leveling concrete to the elevation of the Glen Rose/Twin Mountains Formation contact beneath Units 3 and 4 (785 ft msl - 592 ft msl= 193 ft msl).
5. porosity (η)
   ~~6~~a. Existing fill (large rubble, sand, and gravel) is assumed to have 20% effective porosity (η=0.20).
   b. The Glen Rose limestone is assumed to have 11.9% effective porosity (η=0.119).

RCOL2_02.04.13-05, 06, 07 S01

Table 2.4.13-210
Vertical Migration Hypothetical Pathway Source Term Concentrations

Isotope	Effluent Concentration Limit (ECL) (µCi/gal)[1]	Concentration after 18.75 years of decay (µCi/gal)[2]
H-3	**3.79E+00**	**9.84E+02**
Cr-51	1.89E+00	0.00E+00
Mn-54	1.14E-01	0.00E+00
Fe-55	**3.79E-01**	**8.71E-03**
Fe-59	3.79E-02	0.00E+00
Co-58	7.57E-02	0.00E+00
Co-60	**1.14E-02**	**1.51E-01**
Sr-89	3.03E-02	0.00E+00
Sr-90	**1.89E-03**	**6.36E-03**
Y-91	3.03E-02	0.00E+00
Zr-95	7.57E-02	0.00E+00
Nb-95	1.14E-01	0.00E+00
Ru-103	1.14E-01	0.00E+00
Ru-106	1.14E-02	0.00E+00
Te-129m	2.65E-02	0.00E+00
I-131	3.79E-03	0.00E+00
Cs-134	**3.41E-03**	**6.81E-01**
Cs-136	2.27E-02	0.00E+00
Cs-137	**3.79E-03**	**2.50E+02**
Ce-141	1.14E-01	0.00E+00
Ce-144	1.14E-02	0.00E+00

Notes:

[1]From 10 CFR 20, Appendix B, Table 2

[2]Bold font represents those isotopes that either exceed the ECL or are a few orders of magnitude close to the ECL. Isotopes that are not in bold font are not considered further.

Table 2.4.13-211

RCOL2_02.04.13-05, 06, 07 S01

Source Term Concentration After a Total Decay of 81.53 Years[3]

Isotopes Considered for Sum of Concentration to ECL	Effluent Concentration Limit (ECL) (µCi/gal)[1]	Concentration After Additional 62.75 Years Decay Time[1] - Travel Through Twin Mountains Aquifer to Nearest Property Boundary (µCi/gal)	Curie Content of 105 gallons or 4.0 E5 ml of Volume of Tank Out of 52,800 gallons Released to Environment	Dilution Effect of Twin Mountains Aquifer Prior to Leaving CPNPP Property Boundary (µCi/gal)	Ratio of Activity Concentration (µCi/gal) to ECL (µCi/gal)
H-3	3.79E+00	2.84E+01	3.00E+03	1.50E-02	3.96E-03
Cr-51	1.89E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00
Mn-54	1.14E-01	0.00E+00	0.00E+00	0.00E+00	0.00E+00
Fe-55	3.79E-01	0.00E+00	0.00E+00	0.00E+00	0.00E+00
Fe-59	3.79E-02	0.00E+00	0.00E+00	0.00E+00	0.00E+00
Co-58	7.57E-02	0.00E+00	0.00E+00	0.00E+00	0.00E+00
Co-60	1.14E-02	3.79E-05	4.00E-03	2.00E-08	1.76E-06
Sr-89	3.03E-02	0.00E+00	0.00E+00	0.00E+00	0.00E+00
Sr-90	1.89E-03	1.32E-03	1.39E-01	6.96E-07	3.68E-04
Y-91	3.03E-02	0.00E+00	0.00E+00	0.00E+00	0.00E+00
Zr-95	7.57E-02	0.00E+00	0.00E+00	0.00E+00	0.00E+00
Nb-95	1.14E-01	0.00E+00	0.00E+00	0.00E+00	0.00E+00
Ru-103	1.14E-01	0.00E+00	0.00E+00	0.00E+00	0.00E+00
Ru-106	1.14E-02	0.00E+00	0.00E+00	0.00E+00	0.00E+00
Te-129m	2.65E-02	0.00E+00	0.00E+00	0.00E+00	0.00E+00
I-131	3.79E-03	0.00E+00	0.00E+00	0.00E+00	0.00E+00
Cs-134	3.41E-03	0.00E+00	0.00E+00	0.00E+00	0.00E+00
Cs-136	2.27E-02	0.00E+00	0.00E+00	0.00E+00	0.00E+00
Cs-137	3.79E-03	5.91E+01	6.24E+03	3.12E-02	8.24E+00
Ce-141	1.14E-01	0.00E+00	0.00E+00	0.00E+00	0.00E+00
Ce-144	1.14E-02	0.00E+00	0.00E+00	0.00E+00	0.00E+00

Notes:

[1]Decay time is based upon source term point of entry into Twin Mountains formation reaching nearest property boundary either up-gradient or down-gradient in the shortest amount of time.

RCOL2_02.04.13-05, 06, 07 S01

Table 2.4.13-212
Twin Mountains Formation Groundwater Elevations (2010 Gauging Events)

Well (USGS No.)	Groundwater Elevation (ft msl)
3242403	702.58
3250208	552.49
3242604	538.94
3242904	466.00
3243406	445.72
3243805	394.40

RCOL2_02.0
4.12-8 S01

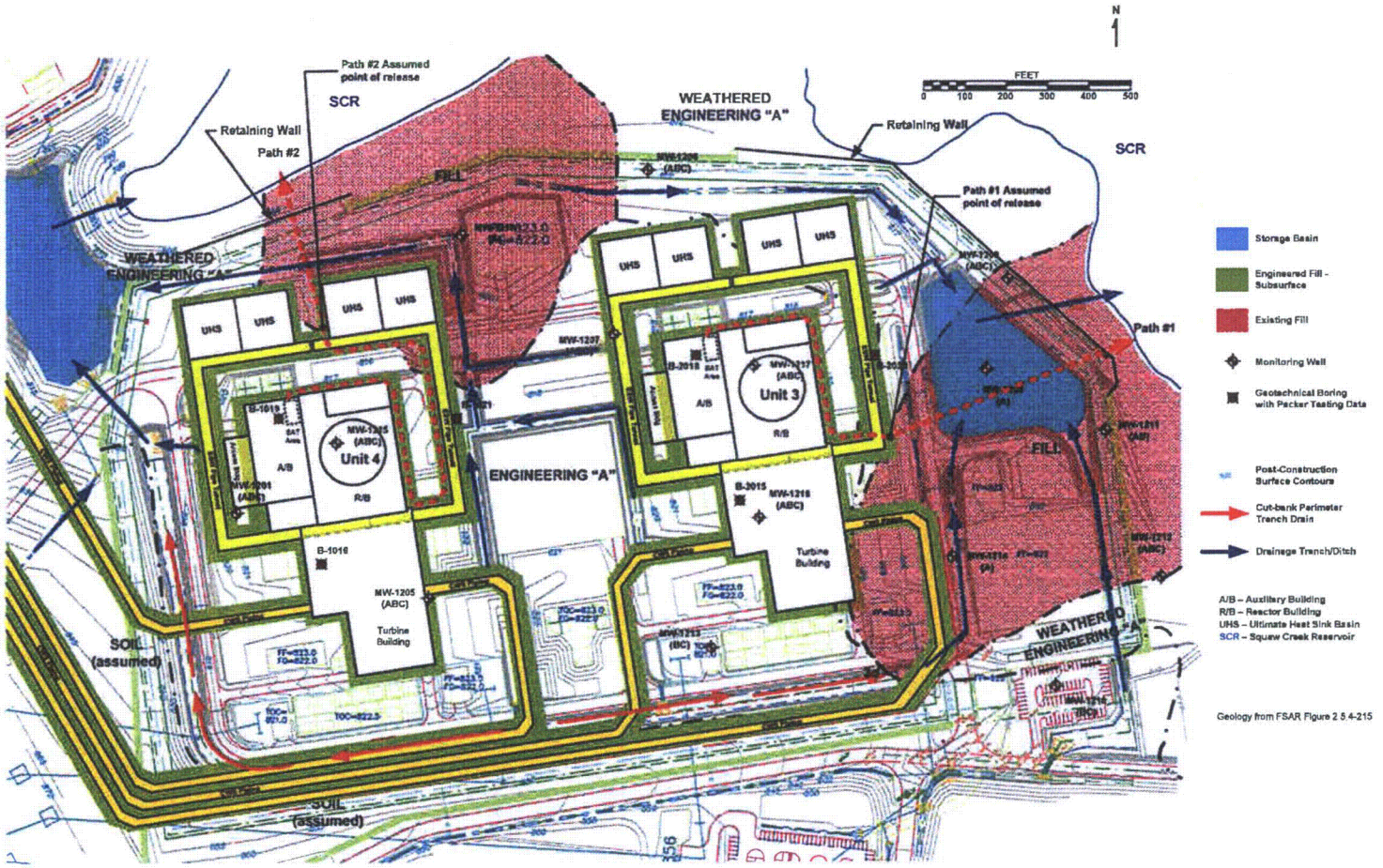


Figure 2.4.12-212 Groundwater Flow Path

RCOL2_02.0
4.12-8 S01

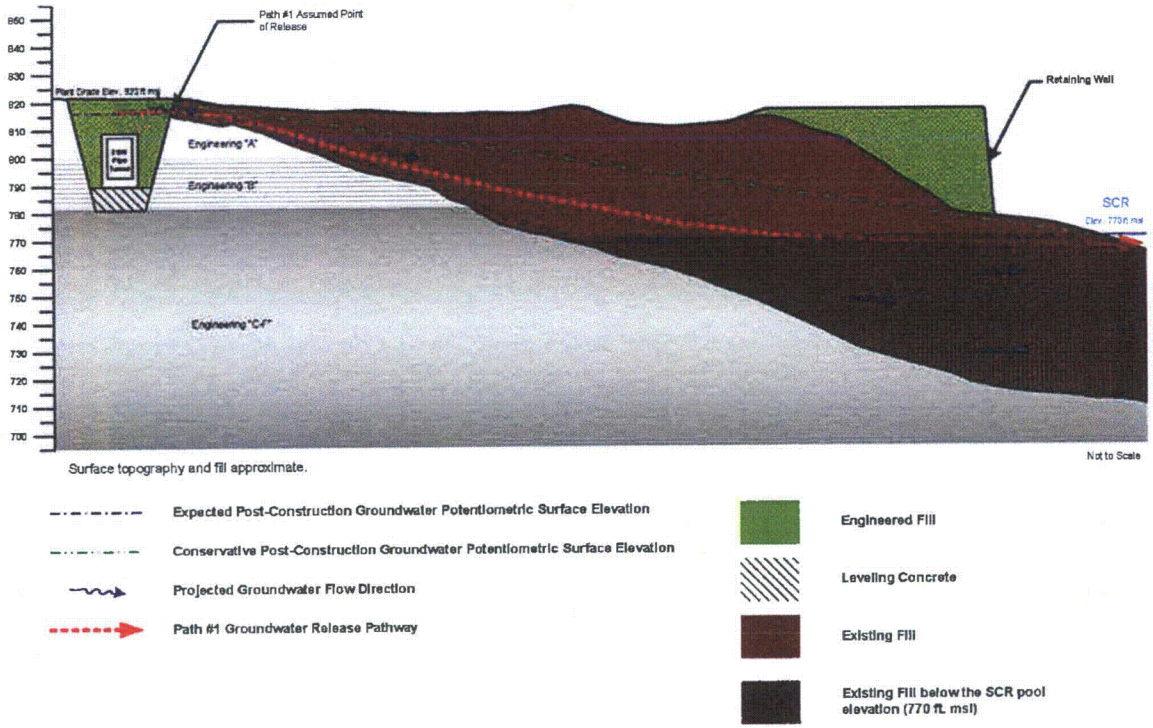


Figure 2.4.12-213 Post Construction Release Flow Path #1

RCOL2_02.04.12-8 S01

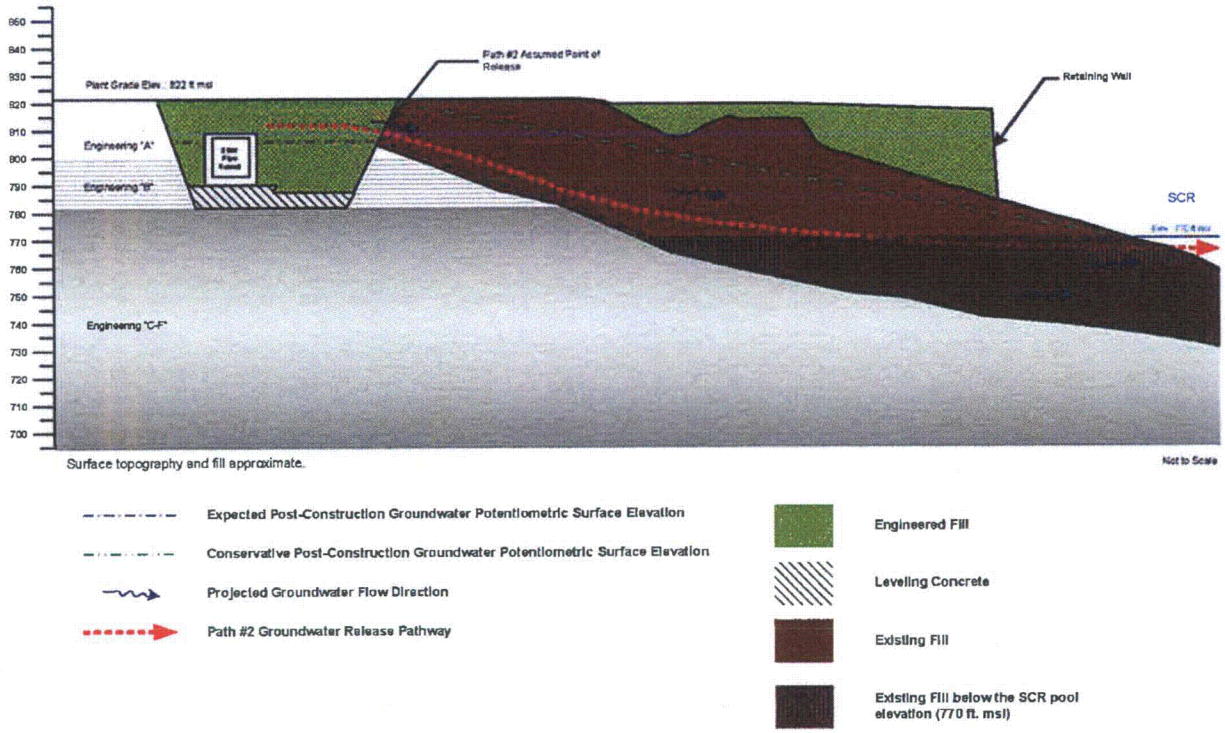


Figure 2.4.12-214 Post Construction Release Flow Path #2

RCOL2_02.0
4.12-8 S01

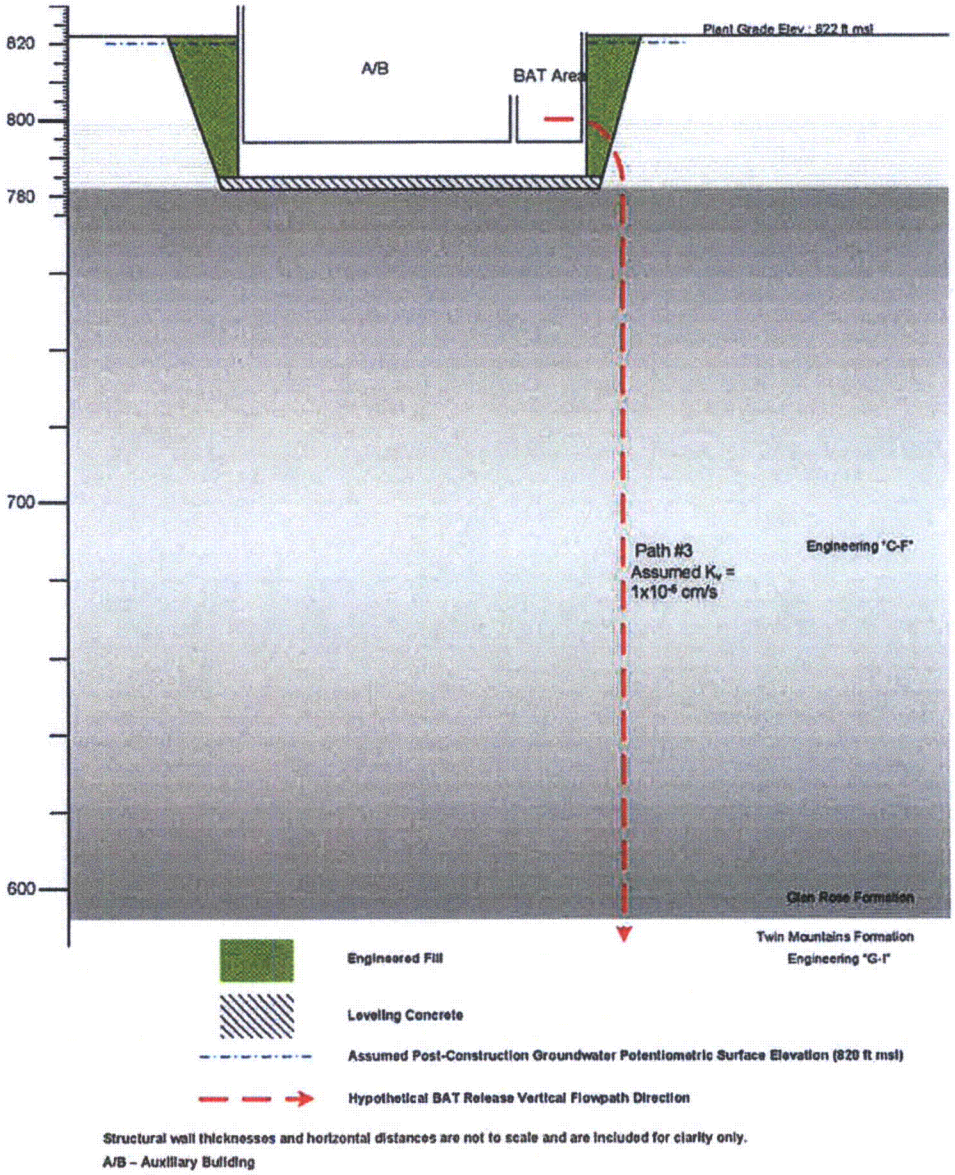


Figure 2.4.12-215 Post Construction Release Flow Path #3

RCOL2_02.0
4.12-8 S01

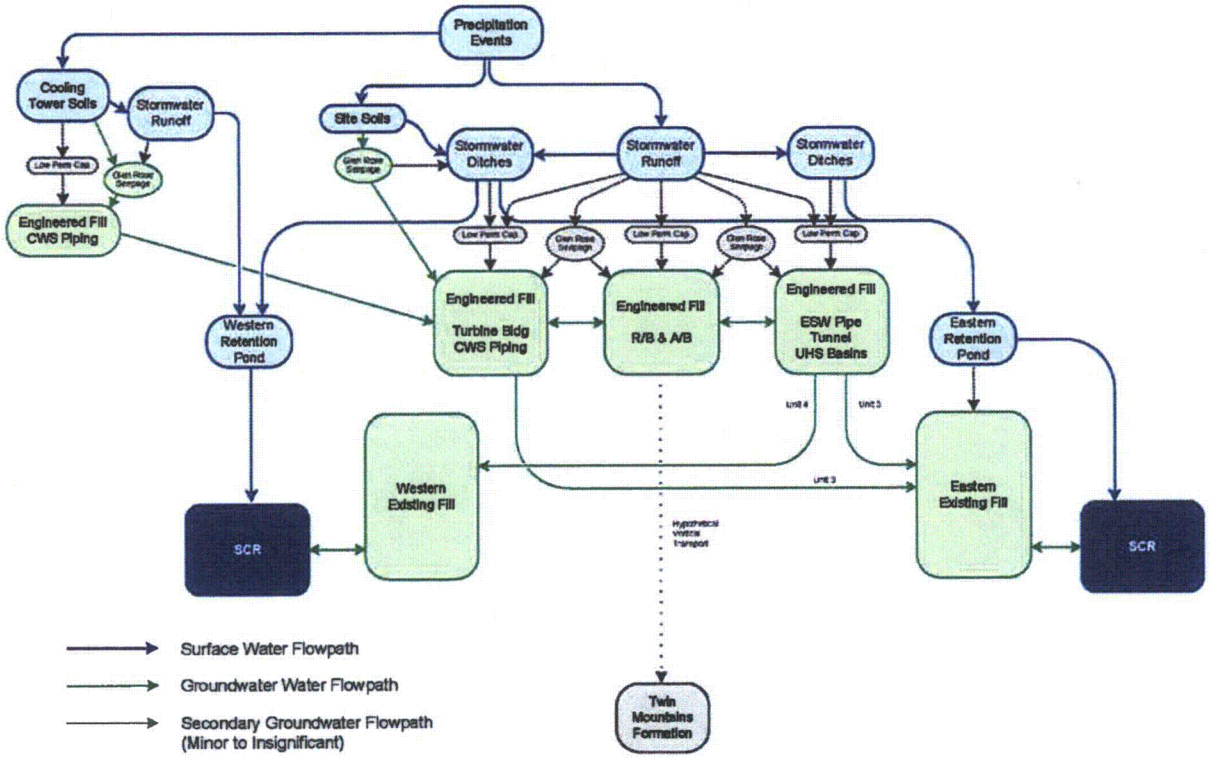


Figure 2.4.12-216 CPNPP Post-Construction Groundwater Conceptual Model

RCOL2_02.0
4.12-8 S01

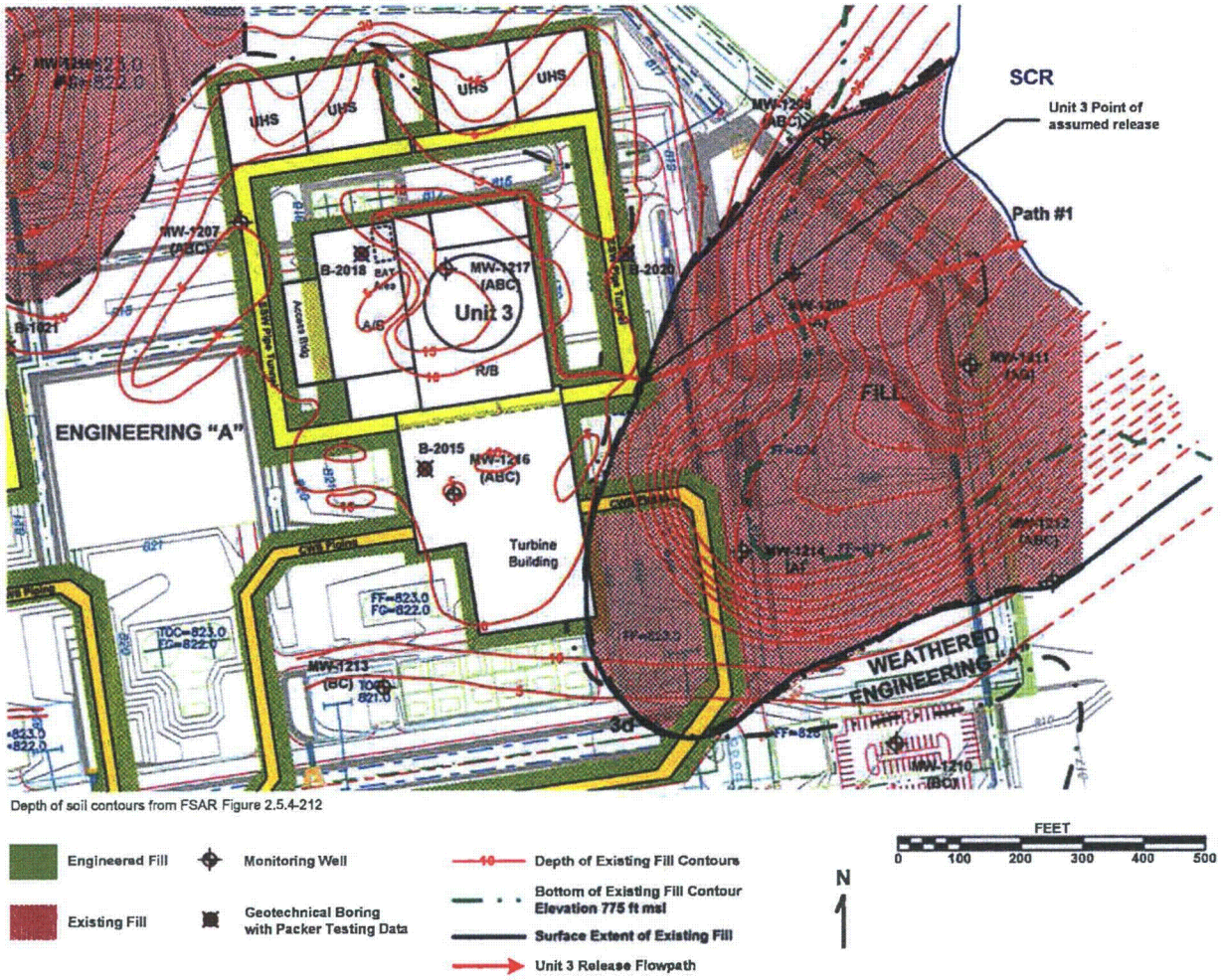


Figure 2.4.13-201 Bounding Horizontal Travel Path for BAT Failure Analysis

Safety-related fill concrete conforms to durability requirements given in Chapter 4 of ACI 349 (Reference 2.5-440). Durability of the fill concrete is assured by the site-specific mix design and by the particular site conditions at CPNPP. The site is located away from the ocean and salt water bodies such that the fill concrete is not exposed to seawater. As stated in Subsection 2.5.1.2.5.9, there are no expansive soils or reactive minerals of appreciable amounts at the site. Therefore, issues related to chemical attack by sulfate, salt attack, or acid attack do not pose concerns for the fill concrete. In addition, CPNPP is located in a relatively warm climate where concerns due to exposure to freeze-thaw action under moist conditions and detrimental effects due to the presence of ice removal agents are insignificant.

The foundation and fill concrete design at CPNPP are such that the issues contained in NRC Information Notice (IN) 97-11 (Reference 2.5-441) are not applicable to fill concrete. No mortar or concrete containing high amounts of calcium aluminate cement is used in foundation or fill concrete. The fill concrete mix design uses Type II Portland cement, consistent with US-APWR DCD Subsection 3.8.4.6.1.1, which is limited to a tricalcium aluminate content of 8% by ASTM C150 (Reference 2.5-482) and is classified by ASTM C150 as moderately resistant to sulfate attack. ~~The maximum anticipated groundwater elevation is at elevation 760 ft, as stated in FSAR Subsection 2.4.1.2.5 and 2.5.4.1.7. This is well below the anticipated bottom of fill concrete.~~ The fill concrete mix design uses fine aggregates, unlike porous concrete consisting only of coarse aggregates and cement. ~~The plant structures are equipped with dampproofing coatings on the sides of below-grade walls and underground drains to collect underground water and channel it away from the structures.~~ The seismic Category I buildings and structures below grade are protected against the effects of flooding,including ground water, by a water barrier on all exterior below-grade concrete members as described in FSAR Subsection 3.4.1.2. Perched water and precipitation run-off do have the potential to come in contact with the fill concrete. However, because of the low groundwater elevation, the use of non-porous fill concrete, and the low amounts of calcium aluminate present in the mix, erosion and leaching concerns and subsequent related effects discussed in IN 97-11 (Reference 2.5-441) are not an issue at CPNPP. Further, FSAR Subsection 3.8.4.7 requires that ground water chemistry be periodically monitored to assure that it remains nonaggressive with respect to concrete structures.

RCOL2_02.05.04-25
RCOL2_02.05.04-11 S01
RCOL2_02.05.04-11 S01

A systematic quality control sampling and testing program ensures that material properties are in compliance with design specifications. Field inspections verify that the required mix is used and that test specimens are collected for testing.

Testing of fill concrete is performed by a qualified testing laboratory that has an established quality assurance program that conforms to NQA-1 requirements. The testing laboratory implements a concrete fill quality control program that includes all aspects of the fill concrete program from the qualification of materials to confirmatory strength testing. Field testing utilizes preapproved procedures that conform to ASTM C31/C31-08a, "Standard Practice for Making and Curing Concrete Test Specimens in the Field~~.~~" (Reference 2.5-483).

RCOL2_02.05.04-25

2.5.4.6.2 Groundwater Occurrence

According to the preliminary results from monitoring of field piezometers within the Units 3 and 4 area, the piezometric levels range between about elevation 775 ft to 858 ft. However, there are also a number of wells that remain dry. Observed piezometric levels are considered to be localized perched water in the upper zone of the Glen Rose Formation, and could possibly be attributed to surface run-off rather than a true indication of permanent groundwater at the site.

As discussed in Subsections 2.4.12 and 2.5.4.1, permanent groundwater occurs deep in the rock mass below plant grade and foundation subgrade elevations. Groundwater inflows into excavations and are therefore not considered to be a significant issue. No significant dewatering or control measures are required during construction excavations. The site grading detail indicates that the potential maximum confined groundwater level within the engineered fill surrounding the main plant area is not expected to exceed elevation 813.5 ft. The groundwater elevation at the site meets US-APWR Key Site Parameter (DCD Table 2.0-1) requirements for maximum groundwater level of 1 ft below plant grade. RCOL2_02.05.04-11 S01 RCOL2_02.05.04-11 S01

2.5.4.6.3 Construction Dewatering

Groundwater, seepage, or runoff, if encountered in open excavations during construction, is anticipated to be of a relatively low volume and may be handled by sumping and pumping. Sumps may be placed within either Glen Rose limestone or sub-foundation concrete that replaces excavated shale materials.

2.5.4.6.4 Groundwater Impacts on Foundation Stability

Because foundations bear directly on limestone with no indication of active karst conditions, as described in Subsection 2.5.1.2.4, or on sub-foundation concrete (that replaces excavated shale materials), the presence of groundwater is not anticipated to significantly impact foundation stability, bearing capacity, or settlement characteristics.

Groundwater or seepage may impact construction activities if water infiltrates shale and claystone materials on excavated side slopes. Shale is likely to deteriorate in the presence of water as a result of excavation and construction traffic that exposes shale surfaces to slaking. Shale materials require removal from trafficked surfaces.

Shale is present at the base of slopes excavated for construction of Units 3 and 4. The surface of shale exposed within the excavated slope is required to be immediately covered by shotcrete or other suitable materials upon completion of excavation to prevent deterioration of shale through exposure to air and/or water.

To minimize the buildup of hydrostatic pressures, adequate drainage ~~for below-grade and~~behind retaining walls and at the base of all fill slopes along the SCR banks is required. Impacts ~~to below grade wall or~~on the retaining wall design and RCOL2_02.05.04-11 S01

performance of the fill slopes along the SCR banks are not significant as long as retaining wall foundation and slope drainage systems perform satisfactorily or the hydrostatic pressure buildup is considered in the design. RCOL2_02.05.04-11 S01

2.5.4.7 Response of Soil and Rock to Dynamic Loading

CP COL 2.5(1) Replace the content of DCD Subsection 2.5.4.7 with the following.

2.5.4.7.1 Overview

This subsection discusses the response of soil and rock to dynamic loading and collection and evaluation of field and laboratory dynamic measurements in order to develop the dynamic site characteristics for seismic design and earthquake engineering purposes. Information presented in Subsections 2.5.1, 2.5.4.1, 2.5.4.2, and 2.5.4.4 form the basis for the dynamic evaluation described herein. The site dynamic properties are used as input for classification of the site in conformance with US-APWR Key Site Parameters (DCD Table 2.0-1), development of the site GMRS presented in Subsection 2.5.2.6.1, and development of FIRS presented in Subsection 2.5.2.6.2. Site dynamic properties also are used for any required SSI analysis as described in Subsection 2.5.2.6.2.

Requirements in 10 CFR Parts 50 and 100 pertaining to site dynamic characterization include:

- An investigation of the effects of prior earthquakes in site soils and rocks including evidence of paleoearthquake liquefaction;
- Field seismic surveys and presentation of interpreted data to develop bounding seismic S-wave and P-wave velocity profiles; and,
- Dynamic laboratory tests on undisturbed samples of foundation soil and rock sufficient to develop strain-dependent modulus reduction and hysteretic damping properties.

All seismic category I and II structures are founded at elevation 782 ft directly on competent and massive Glen Rose Formation Layer C limestone, or thin fill concrete placed over the Layer C limestone. The GMRS and primary FIRS 1 profiles applicable for these conditions are equivalent, and developed at elevation 782 ft at the top of Layer C limestone, as described in Subsection 2.5.2.6. An additional four FIRS profiles (FIRS 2, FIRS 3, FIRS 4_CoV30, and FIRS 4_CoV50) are for specific conditions that are different than the GMRS/FIRS 1 condition. The remaining FIRS are established at plant grade elevation 822 ft and factor combinations of in-place Glen Rose Formation Layers A and B and granular engineered backfill to facilitate evaluation of shallow-embedded plant facilities. The following subsections describe development of the site characteristics used as input for the GMRS and FIRS calculations.

The foundation base mats of all seismic category I and II structures are founded on a limestone layer (engineering Layer C).

The fill materials placed within the excavated areas around Units 3 and 4 and in the north-facing fill slopes are not considered prone to liquefaction ~~for the following reasons:~~because they ~~All fill material~~ consist~~s~~ of engineered compacted fill with a minimum relative compaction of 95 percent (ASTM D1557). The corrected/normalized standard penetration test N-Values are expected to be higher than 30 blows per foot, which is outside the range considered susceptible to soil liquefaction (Reference 2.5-480). RCOL2_02.05.04-11 S01 RCOL2_02.05.04-11 S01

- ~~The engineered compacted fill materials are not in a saturated state. The permanent groundwater table is well below the engineered compacted fill materials.~~

- ~~To minimize any potential for buildup of hydrostatic pressures within the engineered compacted fill, adequate drainage is provided for all below-grade structures and retaining walls, and at the base of all fill slopes.~~

Thus, the engineered compacted fill does not meet the conditions stated in RG 1.206 or RG 1.198 that would cause suspicion of a potential for liquefaction. ~~, and no liquefaction analysis is necessary. Even in the unlikely event that the engineered compacted fill became completely saturated, the soil density is too high and the site PGA range is too low to suspect a potential for liquefaction.~~Liquefaction is therefore not a hazard to CPNPP Units 3 and 4 seismic category I or major plant structures, and the site characteristics meet the US-APWR Standard Design criteria. RCOL2_02.05.04-11 S01

~~Soil liquefaction is also not anticipated within the engineered compacted fill surrounding Units 3 and 4 structures because 1) the permanent groundwater is below the lowest elevation of fill and 2) fill is placed with a high degree of material control and compaction, and 3) the CPNPP site is an area of low seismicity with low GMRS design motions, as described in Subsection 2.5.2.~~ RCOL2_02.05.04-11 S01

2.5.4.9 Earthquake Site Characteristics

CP COL 2.5(1) Replace the content of DCD Subsection 2.5.4.9 with the following.

This subsection briefly summarizes the derivation of the site GMRS and Safe Shutdown Earthquake (SSE) that are detailed in Subsection 2.5.2.6.

The CPNPP Units 3 and 4 site is in a stable continent area with relatively low regional stress and low regional seismicity, as described in Subsections 2.5.1 and 2.5.2, and summarized in Subsection 2.5.4.1. Design ground motions are also relatively low.

A performance-based, site-specific GMRS was developed in accordance with the methodology provided in RG 1.208. This methodology and the GMRS are

3.4 WATER LEVEL (FLOOD) DESIGN

This section of the referenced DCD is incorporated by reference with the following departures and/or supplements.

3.4.1.2 Flood Protection from External Sources

STD COL 3.4(1) Replace the first sentence of the third paragraph in DCD Subsection 3.4.1.2 with the following.

Entrances to all safety-related structures are above the design-basis flooding level (DBFL) listed in Section 2.4, and adequate sloped site grading and drainage prevents flooding caused by probable maximum precipitation (PMP) or postulated failure of non safety-related, non seismic storage tanks located on site.

CP COL 3.4(5) Replace the fourth paragraph in DCD Subsection 3.4.1.2 with the following.

No site-specific flood protection measures such as levees, seawalls, floodwalls, site bulkheads, revetments, or breakwaters are applicable at CPNPP Units 3 and 4, since the plant is built above the DBFL and has adequate site grading. The lowest point of the structure foundation is above the groundwater elevation identified in Section 2.4, and therefore no permanent dewatering system is required.

CP COL 3.4(4) Replace the seventh paragraph in DCD Subsection 3.4.1.2 with the following.

All seismic Category 1 buildings and structures below-grade are protected against the effects of flooding, including ground water. This protection is achieved by providing a water barrier on all exterior below-grade concrete members. The water barrier consists of providing waterstops at all below-grade construction joints in the exterior wall and base mats subjected to ground water seepage, and membrane waterproofing material at all below-grade exterior wall surfaces. The foundation slab water barrier system consists of crystalline waterproofing compound applied between the base mat and fill concrete/bedrock. The compound will either be spray applied or dry-shake to the fill concrete/bedrock. ~~The lowest point of the structure foundation is above the groundwater elevation identified in Section 2.4. In addition, no intermittent head of water occurs from surface precipitation or groundwater due to the placement of course aggregate wrapped in geotextile filter fabric with perforated drainage pipe sloped to daylight to Squaw Creek Reservoir. Construction joints in the exterior walls and base mats are provided with water stops to prevent seepage of ground water. A~~ RCOL2_02.05.04-11 S01

~~dampproofing barrier treatment that resists the passage of ground water in the absence of hydrostatic pressure is therefore applied to all subgrade outer foundation walls in accordance with American Concrete Institute (ACI) 515.1R-79 (Reference 3.4-201). A cementitious membrane waterproofing~~A cementitious membrane coating made out of a crystalline waterproofing compound is provided on the inside face of the UHS basin outermost walls and foundation slab, including the UHS sump pit, to prevent water migration from the UHS basin into the subgrade. RCOL2_02.05.04-11 S01

STD COL 3.4(3) Replace the last sentence in the ninth paragraph in DCD Subsection 3.4.1.2 with the following.

Site-specific potential sources of external flooding such as the cooling tower, service water piping, or circulating water piping are not located near structures containing safety-related SSCs, with the exception of piping entering plant structures. The CWS enters only within the T/B, and any postulated pipe break is prevented from back-flowing into the safety-related R/B by watertight separation. Postulated pipe breaks near structures are prevented from entering the structures by adequate sloped site grading and drainage.

3.4.1.3 Flood Protection from Internal Sources

STD COL 3.4(7) Replace the last sentence in the last paragraph of DCD Subsection 3.4.1.3 with the following.

Three site-specific safety-related structures have been evaluated for internal flooding concerns: the UHSRS, the ESWPT, and the PSFSV. Other site-specific buildings and structures in the plant yard are designated as non safety-related. By definition, their postulated failure due to internal flooding or other postulated events do not adversely affect safety-related SSCs or required safety functions.

Each of these three structures is configured with independent compartments, divisionally separated. Internal flooding of any one compartment and corresponding division will not prevent the system from performing required safety-related functions. Postulated flooding events such as those caused by moderate energy line break (MELB) or fire suppression system activation within one division will affect that respective division only. Flooding affecting one compartment will not affect adjacent areas.