Dominion Nuclear North Anna, LLC
5000 Dominion Boulevard, Glen Allen, VA 23060

Dominion®

March 3, 2005

U. S. Nuclear Regulatory Commission
Attention: Document Control Desk
Washington, D.C. 20555

Serial No. 05-785B
ESP/JDH
Docket No. 52-008

DOMINION NUCLEAR NORTH ANNA, LLC
NORTH ANNA EARLY SITE PERMIT APPLICATION
RESPONSES TO DRAFT SAFETY EVALUATION REPORT OPEN ITEMS

On December 20, 2004, the NRC issued its Draft Safety Evaluation Report (DSER) for Dominion Nuclear North Anna, LLC's North Anna Early Site Permit application. The DSER contained open items for which the NRC requested a response by March 3, 2005.

A response to DSER Open Item 2.5-1 was provided in Dominion's January 25, 2005 letter, Serial No. 04-785. A planned approach for responding to Open Item 2.5-2 was submitted in Dominion's February 18, 2005 letter, Serial No. 04-785A.

This letter contains responses to the following DSER open items:

2.3-1, 2.3-2, 2.3-3, 2.3-4, 2.4-1, 2.4-2, 2.4-3, 2.4-4, 2.4-5, 2.4-6, 2.4-7, 2.4-8, 2.4-9, 2.4-10, 2.4-11, 2.5-2 (partial response), 13.3-3 and 13.3-6

A response to Open Item 2.1-1 will be provided separately. Also, as stated in our February 18, 2005 letter, the remaining response to Open Item 2.5-2 will be submitted by March 31, 2005.

On February 23, 2005, Dominion met with the NRC staff to discuss the open items and other items related to the DSER. The status of several open items related to emergency preparedness was discussed. Dominion had previously provided information for Open Items 13.3-1, 13.3-2, 13.3-4, 13.3-5, 13.3-7, 13.3-8, 13.3-9, and 13.3-10 on October 20, 2004. The information was received too late for consideration by the NRC staff in the DSER. However, at the February 23, 2005 meeting, the NRC staff indicated that the responses were adequate and that no additional information for the open items was necessary.

Another matter discussed during the meeting involved early site permit conditions and action items. In Dominion's view, objective criteria should be established upon which permit conditions and action items would be based. Dominion continues to be interested in working with the staff to develop such criteria as well as continuing a dialog on the proposed permit conditions and action items for the North Anna early site permit.

Finally, it is our intent to update the North Anna ESP application to reflect our responses to the DSER open items. Planned changes to the application are identified following the response to each open item.

If you have any questions or require additional information, please contact Mr. Joseph Hegner at 804-273-2770.

Very truly yours,

Eugene S. Grecheck
Vice President-Nuclear Support Services

Enclosure: Responses to Draft Safety Evaluation Report Open Items.

Commitments made in this letter:

1. Update the North Anna ESP application to reflect responses to DSER Open Items.

cc: (with enclosure)

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COMMONWEALTH OF VIRGINIA

COUNTY OF HENRICO

The foregoing document was acknowledged before me, in and for the County and Commonwealth aforesaid, today by Eugene S. Grecheck, who is Vice President, Nuclear Support Services, of Dominion Nuclear North Anna, LLC. He has affirmed before me that he is duly authorized to execute and file the foregoing document on behalf of Dominion Nuclear North Anna, LLC, and that the statements in the document are true to the best of his knowledge and belief.

Acknowledged before me this 3rd day of March, 2005.

My Commission expires: May 31, 2006

Vicki L. Hull
Notary Public

(SEAL)

Enclosure

Responses to Draft Safety Evaluation Report Open Items

DSER Open Item 2.3-1 (DSER page 2-30)

> The applicant proposed a design-basis site characteristic wind speed of 64 mi/h, which the applicant stated represents a "fastest mile of wind" at 10 m (33 ft) above the ground with a 100- year return period. This value is presented in Table A-7 of ANSI A58.1-1982, "Minimum Design Loads for Buildings and Other Structures," as the extreme fastest-mile wind speed having a 0.01 annual probability of being exceeded at Richmond. The applicant's chosen 100-year return period fastest-mile design-basis wind speed of 64 mi/h is not conservative when compared to the minimum 50-year return period fastest-mile basic wind speed of 70 mi/h specified in Section 6.5.2 of ANSI A58.1-1982. The applicant's chosen value is also not conservative when compared to the highest fastest-mile wind speed of 68 mi/h recorded at Richmond during the 32-year period of record, 1958–1989. The applicant needs to justify an acceptable design basis wind speed. This is **Open Item 2.3-1.**

<u>Response</u>

SSAR Section 2.3.1.3.1 discusses extreme winds at the ESP site and indicates that according to ANSI A58.1-1982, the 100-year return value for the fastest-mile-wind speed at Richmond, Virginia is 64 miles per hour. This value was calculated based on actual wind speed data collected from the National Weather Service (NWS) station in Richmond over a 27-year period prior to 1979. This value was reported in the SSAR in accordance with Section II.4 of the NRC's draft ESP Review Standard (RS-002, Reference 1) to provide a conservative characteristic of the site. In addition, a wind speed of 68 miles per hour, observed in October 1954, was identified to acknowledge that it is possible for there to be specific occasions when the actual wind speed exceeds the calculated 100-year return value. The 70 mile-per-hour wind speed cited in the DSER statement of the Open Item above is the value specified in ANSI A58.1-1982 for a fastest-mile-wind speed that is used in the calculation of wind load when designing buildings and other structures. The design or design basis of buildings and structures is not addressed in the ESP application.

In RAI 2.3.1.1 a) (Reference 2), the NRC requested a site characteristic value for a 100-year return 3-second gust wind speed. Based on data and guidance in ASCE 7-02, the wind speed corresponding to a 100-year return 3-second gust was determined to be 96 miles per hour. This value was provided in the response to RAI 2.3.1.1 a) (Reference 3) and incorporated in SSAR Section 2.3.1.3.1. Thus, this 96 mile-per-hour 100-year return 3-second gust will be identified in Table 1.9-1 as a conservative site characteristic for basic wind speed.

References

1. Draft RS-002, "Processing Applications for Early Site Permits," U.S. Nuclear Regulatory Commission, August 11, 2003.

2. March 8, 2004 Letter from Michael L. Scott, Dominion ESP Project Manager, U.S. Nuclear Regulatory Commission, to Mr. David A. Christian, Dominion, "Request for Additional Information Letter No. 1.

3. August 2, 2004 Letter from Eugene S. Grecheck, Vice President-Nuclear Support Services, Dominion, to U.S. Nuclear Regulatory Commission, Document Control Desk, "Dominion Nuclear North Anna, LLC, North Anna Early Site Permit Application, Response to Request for Additional Information No. 4."

Application Revision

The Basic Wind Speed site characteristic in SSAR Table 1.9-1 will be revised to read as follows:

Item	Single Unit/Group Value [Second Unit/Group Value]	Description and References
Basic Wind Speed	96 mph [Same for 2nd unit/group]	▪ 3-second gust wind velocity, associated with a 100-year return period, at 33 feet (10 meters) above ground level in the site area. ▪ Refer to Section 2.3.1.3.1.

SSAR Section 2.3.1.3.1 will be revised to read as follows:

> According to American National Standard, ANSI A58.1-1982, the operating basis wind velocity at 33 feet (10 meters) above ground level in the ESP site area associated with a 100-year return period is 64 miles per hour (mph) (Reference 45). The fastest-mile-wind speed is defined as the passage of one mile of wind with the highest speed for the day. The actual observed fastest-mile-wind speed at Richmond (68 miles per hour) was recorded at that station in October 1954 (Reference 6). The 3-second gust wind speed that represents a 100-year return period is 96 mph at 10 meters above ground. This wind speed was determined in accordance with the guidance in Reference 46, and is selected as a conservative basic wind speed site characteristic.

SSAR Table 2.3-4 will be deleted.

DSER Open Item 2.3-2 (DSER page 2-31)

The applicant has identified a 100-year return period snowpack of 30.5 lbf/ft2 for the North Anna ESP site. The applicant determined this value in accordance with the guidance of SEI/ASCE 7-02. Because the applicant performed its analysis in accordance with the appropriate guidance and the results bound the observations described above, the staff concludes that a 100-year return period snowpack site characteristic value of 30.5 lbf/ft2 is acceptable.

The applicant has identified a 48-hour winter probable maximum precipitation (PMP) value of 20.75 in. for the North Anna ESP site. The winter PMP value is specified in RG 1.70 to assess the potential snow loads on the roofs of safety-related structures. However, the applicant has proposed an alternative approach (as discussed in the following paragraph) for defining the site characteristic snow load that does not rely on the winter PMP value. Consequently, the staff did not evaluate or accept the applicant's winter PMP value.

As noted above, the applicant has proposed a site characteristic ground snow load value of 30.5 lbf/ft2, which is based on the 100-year return period snowpack for the North Anna ESP site. Section 2.3.1.2 of RG 1.70 states that the weight of snow and ice on the roof of each safety-related structure should be a function of the weight of the 100-year return period snowpack and the weight of the 48-hr winter PMP for the site vicinity. The combined 100-yr return snowpack and the estimated winter PMP may be an unreasonable snow/ice roof loading for a structure at the North Anna ESP site, given that snow generally remains on the ground for only 1 or 2 days. As an alternative, a combination of the 100-year return snowpack and the maximum-recorded monthly snowfall in the North Anna ESP site region may be a reasonably conservative site-characteristic ground snowload for designing the roofs of safety-related structures. The applicant needs to justify the exclusive use of snowpack weight or provide an alternative method. This is **Open Item 2.3-2**.

Response

Dominion does not agree with the characterization in the DSER that the snowpack weight is being used exclusively to determine roof load. As stated in SSAR Section 2.3.1.1, design basis snow load conditions are characterized based on the weights of the probabilistic snow pack and winter PMP amounts. In response to RAI 2.3.1-1 e) (Reference 1) the weight of the 100-year return period snowpack (30.5 pounds per square foot) and the amount of the 48-hour winter PMP (inches of precipitation) were provided. These values were included in SSAR Sections 2.3.1.3.4 and 2.4.7.6 as part of Revision 3 to the ESP application.

During the February 23, 2005 public meeting, the NRC requested that Dominion consider the guidance contained in Reference 2 in responding to this Open Item. The NRC also requested that Dominion provide substantiation of the form of the 48-hour winter PMP (i.e., is the 48-hour winter PMP liquid or frozen precipitation). As described in the response to RAI 2.3.1-1 e), the 48-hour winter PMP was linearly interpolated from values shown in Figures 35 and 45 of NUREG/CR-1486 (Reference 3) for the 24-hour and 72-hour, respectively, events in December. The Abstract in NUREG/CR-1486 states that, "Estimates of the upper limit to rainfall [emphasis added] that the atmosphere can produce (probable maximum precipitation) are given in this study for durations from 6 to 72 hours for each month of the year for 10 mi² areas." Section 2.1, Background, states, "As in all PMP studies, basic data are the extreme record storm rainfalls [emphasis added]." Thus, it is concluded that the 48-hour winter PMP site characteristic is in the form of rainfall.

As a result of discussions with the NRC, an additional 48-hour maximum winter snowfall event will be added to the 100-year return period snowpack as a site characteristic. This additional 48-hour winter snowfall event has been conservatively determined from the regional summary data presented in SSAR Table 2.3-5. As noted in this table, the maximum 24-hour snowfall for the region was recorded at Richmond in January 1940 (snowfall actually occurred over two calendar days). In order to conservatively encompass a 48-hour maximum winter snowfall, the maximum monthly snowfall from Table 2.3-5 for Richmond, 28.5 inches, will be used. As discussed in SSAR Section 2.4.7.3, assuming a snow density of 0.1, 28.5 inches of snow is equivalent to 2.85 inches of water, which is equivalent to a ground snow load of approximately 15 pounds per square foot. When added to the snowpack ground load of 30.5 pounds per square foot, a maximum ground load of 45.5 pounds per square foot would be realized from this combination.

The features of a specific roof design would dictate the proper combination of the winter precipitation loads on that roof surface. As noted in Section 2.4.7 of the NRC's draft ESP Review Standard (RS-002, Reference 4), applicants would need to demonstrate that structures, systems and components important to safety are designed to withstand the effects of natural phenomena (GDC 2) at the COL stage, not at the ESP stage. SSAR Section 2.4.7.6 acknowledges the requirements of RG 1.70 and states:

> The maximum load experienced by the roof structure, due to precipitation, is dependent on the roof design/configuration. For example, the roof load could be governed by the maximum accumulation of snow and a surcharge due to the loading from the overflow depth as runoff flows over the roof. The design capacity of the roof structure, and possibly other design features, which demonstrate acceptable roofing structure performance for the selected reactor design, would be described in the COL application.

References

1. August 2, 2004 Letter from Eugene S. Grecheck, Vice President-Nuclear Support Services, Dominion, to U.S. Nuclear Regulatory Commission, Document Control Desk, "Dominion Nuclear North Anna, LLC, North Anna Early Site Permit Application, Response to Request for Additional Information No. 4."

2. NRC Memorandum, "Site Analysis Branch Position – Winter Precipitation Loads," from Harold R. Denton, Assistant Director for Site Safety, Division of Technical Review, Office of Nuclear Reactor Regulation, to R. R. Maccary, Assistant Director for Engineering, TR, March 24, 1975.

3. NUREG/CR-1486, "Seasonal Variation of 10-Square-Mile Probable Maximum Precipitation Estimates, United States East of the 105th Meridian, Hydrometeorological Report No. 53," U.S. Nuclear Regulatory Commission, U.S. Department of Commerce, National Oceanic and Atmospheric Administration, April 1980.

4. Draft RS-002, "Processing Applications for Early Site Permits," U.S. Nuclear Regulatory Commission, August 11, 2003.

Application Revision

SSAR Table 1.9-1 will be revised to establish a "Winter Precipitation" category heading in place of the existing "Snow Load" category. The Winter Precipitation category will include three sub-items as separate characteristics (100-year Snowpack, 100-year Snowpack plus 48-hour Maximum Snowfall, 48-hour Winter PMP) as identified below:

Item	Single Unit/Group Value [Second Unit/Group Value]	Description and References
Winter Precipitation		
▪ 100-year Snowpack	30.5 lb/sq ft [Same for 2nd unit/group]	▪ Weight, per unit area, of the 100-year return period snow pack at the site. ▪ Item 1.2.2 of Table 1.3-1 ▪ Refer to Section 2.3.1.3.4 & Section 2.4.7.6

Item	Single Unit/Group Value [Second Unit/Group Value]	Description and References
▪ 100-year Snowpack plus 48-hour Maximum Snowfall	45.5 lb/sq ft [Same for 2nd unit/group]	▪ 48-hour maximum snowfall (28.5 inches, ~ 15 lb/sq ft) on top of a 100-year return snowpack (30.5 lb/sq ft). ▪ Item 1.2.2 of Table 1.3-1 ▪ Refer to Section 2.3.1.3.4 & Section 2.4.7.6
▪ 48-hour Winter PMP	20.75 inches [Same for 2nd unit/group]	▪ Maximum probable winter rainfall in 48-hour period. ▪ Item 1.2.2 of Table 1.3-1 ▪ Refer to Section 2.3.1.3.4 & Section 2.4.7.6

DSER Open Item 2.3-3 (DSER page 2-33)

> The staff believes that the applicant needs to identify an additional UHS design-basis site characteristic for use in evaluating the potential for water freezing in the UHS water storage facility, a phenomenon which would reduce the amount of water available for use by the UHS. The lowest 7-day average air temperature recorded in the site region may be a reasonably conservative site-characteristic for evaluating the potential for water freezing in the UHS water storage facility. This item is unresolved and is **Open Item 2.3-3.**

Response

In the DSER Open Item, the NRC requested that Dominion propose an additional UHS design-basis site characteristic that would be used in the evaluation of the potential for ice formation in the UHS and suggested using the lowest 7-day average air temperature recorded in the site region.

While the lowest 7-day average air temperature may provide some indication of possible ice formation in the UHS, there does not appear to be sufficient technical or regulatory basis to propose it as a site characteristic. Our assessment is that it may yield non-conservative results if used in an evaluation of the maximum thickness of ice on the surface of the UHS basin.

Instead, as a result of discussions with the NRC at the February 23, 2005 public meeting, it is proposed to use the maximum-cumulative-degree-days-below-freezing as the site characteristic. This is the same parameter that was used in the evaluation of possible ice thickness on Lake Anna in SSAR Section 2.4.7.5. The parameter is measured in degree-days.

In DSER Section 2.4.7, the NRC described its independent evaluation of possible ice formation on Lake Anna. In the DSER evaluation, mean daily air temperatures from the Piedmont Research Station were used. The calculation of cumulative degree-days-below-freezing started on December 1st each year and ran through the following March 31st, and the maximum value was selected from this period. The DSER evaluation found that a maximum of 321.8 cumulative degree (F)-days-below-freezing occurred during the December 1, 1976 to March 31, 1977 period. Using daily temperature data for the Piedmont Station from NCDC (Reference 1), Dominion was able to essentially reproduce the DSER results.

During detailed engineering, should a reactor design be selected that would require a conventional UHS, the design of the towers and the basin would consider the possible formation and buildup of ice as determined using the maximum cumulative degree (F)-

days-below-freezing. The DSER value of 322 degree (F)-days, based on mean temperature data from the Piedmont Research Station and a simple integration for the period from December 1st through March 31st, will be identified as a site characteristic in SSAR Table 1.9-1.

The Piedmont meteorological data would be used for this site characteristic only, and its use here does not undermine the use of Richmond data to adequately represent the ESP site.

References

1. *Cooperative Summary of the Day*, TD3200, Period of Record through 2001 includes daily weather data from the Eastern United States, Puerto Rico, and the Virgin Islands, data released November 2002, Version 1.0 (CD-ROM), data listings for Charlottesville 2W, Fredericksburg National Park, Gordonsville 3S, Louisa, Partlow 3WNW, Piedmont Research Station, Bremo Bluff PWR and Free Union, Virginia, NCDC, NOAA.

Application Revision

SSAR Table 1.9-1 will be revised to add the following new site characteristic in the category of "Ultimate Heat Sink Ambient Air Controlling Parameters":

Item	Single Unit/Group Value [Second Unit/Group Value]	Description and References
▪ Maximum-Cumulative-Degree-Days-Below-Freezing	322 degree (F)-days [Same for 2nd unit/group]	▪ Meteorological condition resulting in the maximum formation of surface ice in the UHS basin ▪ Refer to Section 2.3.1.3.8

SSAR Section 2.3.1.3.8 will be revised to include the following as the final paragraph:

> The meteorological conditions resulting in the maximum formation of surface ice (and therefore the minimum initial volume of liquid water available for cooling) is the cumulative dry bulb temperature depression below freezing, measured in degree-days. This is determined by integrating over time, from December 1st of any given year through the following March 31st, the depression below freezing of the daily mean dry bulb temperature using the meteorological data for the Piedmont Research Station (Reference 40). The maximum-cumulative-degree-days-below-freezing was determined to be approximately 322 degree (F)-days, and it occurred in the December 1976 – March 1977 period.

SSAR Section 2.4.7.5 will be revised to read as follows:

> The formation of a surface ice sheet in a cooling water lake can exert forces on the contact structures due to ice expansion or to the drag force caused by wind acting on unrestrained ice sheets.
>
> Shoreline intakes designed with approach channels can become obstructed by ice jams. This is possible at lake intakes where wind may drive the ice toward the shoreline. However, trash racks prevent the entry of large pieces of ice broken from ice sheets (Reference 28).
>
> However, if all of the existing and new units were off-line during a relatively sustained freezing weather period, the formation of surface ice is possible, based on examination of the mean daily air temperature for the 1961–1995 time period. The data show that there were several years in which the mean daily temperature in the December through March time frame was below freezing for one to three weeks.
>
> The maximum ice thickness that could have formed under historic low air temperatures with no units in service has been predicted. The meteorological data for the Piedmont Research Station (Reference 61) have been analyzed to determine the degree-days below freezing. In the December 1976 through March 1977 period, there were about 322 cumulative degree-days below freezing. Using this information and employing Assur's method as presented in Chow (Reference 29) (Reference 30), the calculated ice thickness is approximately 17.1 inches. This ice layer would not impact water flow upon restart due to the water depth at the new intakes (a minimum of approximately 24 feet). Instead, this surface ice layer would insulate and provide protection against the formation of frazil ice. However, the formation of surface ice can exert a high load on the intake structure wall in contact with the water. Ice forces would be accounted for

in the design of the intake and described in the COL application. It should also be noted that the intakes and associated pumps for the new units would not be safety-related facilities. Emergency cooling and service water needed to maintain the new units in a safe mode would be supplied by a separate UHS. Therefore, no safety-related facilities would be affected by ice layer formation on the lake.

Upon restart of the units and the circulation of warm water, the ice would gradually melt and break. The velocity induced by the flow can cause some of the ice floes to be withdrawn or moved by the water (Reference 31). Although the design of the intake has not been developed to enable the determination of ice floe size that might be withdrawn, the presence of trash racks and traveling screens would prevent such ice from reaching the pumps. The accumulation of ice at the trash racks and traveling screens could clog them and reduce the flow capacity of the intake structure. However, since emergency cooling and service water would be provided by the UHS, no safety-related facilities would be affected by ice floe accumulation on the lake.

Section 2.4 References will be revised to add the following new reference:

61. *Cooperative Summary of the Day*, TD3200, Period of Record through 2001 includes daily weather data from the Eastern United States, Puerto Rico, and the Virgin Islands, data released November 2002, Version 1.0 (CD-ROM), data listings for Charlottesville 2W, Fredericksburg National Park, Gordonsville 3S, Louisa, Partlow 3WNW, Piedmont Research Station, Bremo Bluff PWR and Free Union, Virginia, NCDC, NOAA.

SSAR Table 2.4-13 will be deleted.

DSER Open Item 2.3-4 (DSER page 2-40)

> Because of the limited and localized nature of the expected terrain modifications associated with the development of the ESP facility, the staff finds that these terrain modifications, along with the resulting plant structures and associated improved surfaces, will not have enough of an effect on local meteorological conditions to affect plant design and operation. Similarly, because the operation of an open-cycle cooling system for the applicant's proposed unit 3 is not expected to significantly impact either atmospheric temperature extremes or increase the occurrence of local fog, the staff finds that the atmospheric impact of the operation of an open-cycle cooling system for proposed unit 3 will not affect plant design and operation. However, the applicant has not described how potential increases in atmospheric temperature resulting from the operation of closed-cycle dry cooling towers associated with proposed unit 4 would impact plant design and operation. This item is unresolved and is **Open Item 2.3-4.**

<u>Response</u>

Normal plant cooling for a new Unit 4 would use a closed-cycle dry cooling system. In this system, exhaust from the plant's steam turbines would be directed to a surface condenser where the heat of condensation would be rejected to a closed loop of cooling water. The heated cooling water would be circulated to the finned tubes of dry cooling towers where the heat content of the cooling water would be conductively transferred to the ambient air. The dry cooling towers would be located in the cooling tower area shown on SSAR Figure 1.2-4. The dry towers would be approximately 150 feet high and would consist of a series of modules, each containing air circulating fans.

The response to RAI 2.3.2-1 (Reference 1) explained that the convective and conductive heat losses to the atmosphere resulting from the operation of the Unit 4 closed-loop dry tower system would dissipate rapidly due to continuous mixing and entrainment of the surrounding air with the warm air plume. Thus, it was concluded that any increases in overall ambient temperature would be small and localized to the North Anna site and would not affect ambient atmospheric or ground temperatures beyond the site boundary.

Similarly, the operation of the dry cooling towers would be expected to have minimal impact on the design and operation of new Units 3 and 4 for the following reasons:

- The warm air plume from the dry cooling towers would tend to rise vertically, initially driven by the velocity imparted by the fans, and continuing due to the thermal buoyancy. During most expected atmospheric conditions, wind that would alter the course of the plume rise would also tend to enhance the mixing in

of cooler ambient air, thereby enhancing dispersal of the heated plume. Only a strong wind, blowing across the bank of cooling towers, could cause a plume downwash due to building wake effect. The strong wind would also have the effect of increasing the mixing with cooler air from outside the plume. Therefore, the change in temperature at ground level would be negligible.

- The prevailing summertime wind direction is from the south and southwest. This would tend to carry the warm plume from the towers away from the plant.

- At its closest point, the plant envelope area is over 300 feet away from the cooling tower area as shown in SSAR Figure 1.2-4. The top of the cooling towers would be approximately 150 feet above grade, and the top of the tallest power block structure in the plant envelope area would be about 234 feet above grade. Any warm air exiting from the cooling towers would be expected to rise more than the difference in height between the structures, as it traverses the more than 300-foot distance that separates them.

Since the specific design of the plant is not known at this time, it is not possible to predict with certainty the warm air transport/dispersion from the cooling tower to specific plant features, such as HVAC air intakes. The potential impact on the design and operation of the new unit(s) from any cooling-tower-induced increase in the local ambient air temperature would be considered as part of detailed engineering.

Reference

1. August 2, 2004 Letter from Eugene S. Grecheck, Vice President-Nuclear Support Services, Dominion, to U.S. Nuclear Regulatory Commission, Document Control Desk, “Dominion Nuclear North Anna, LLC, North Anna Early Site Permit Application, Response to Request for Additional Information No. 4.”

Application Revision

The last paragraph of SSAR Section 2.3.2.3 will be revised to read as follows:

> Similarly, the convective and conductive heat losses to the atmosphere resulting from operation of the Unit 4 closed-loop dry tower system would dissipate rapidly through continuous mixing and entrainment with the surrounding moving air mass. Therefore, any increases in overall ambient temperature would be very localized to the NAPS site and would not affect the ambient atmospheric and ground temperatures beyond the NAPS site boundary, or otherwise significantly alter local temperature patterns. The potential impact on the design or operation

of the new unit(s) from any cooling-tower-induced increase in the local ambient air temperature would be considered as part of detailed engineering.

DSER Open Item 2.4-1 (DSER pages 2-61 and 2-62)

The applicant's response to RAI 2.4.1-1 included a figure that listed the coordinates of the corners of the ESP PPE (ESP site footprint). However, the applicant did not identify the coordinate system. The staff needs information regarding the coordinate reference system and the units of these coordinates to fully define the boundaries of the ESP site footprint. This is **Open Item 2.4-1**.

Response

Figure 1 in the response to RAI 2.4.1-1 a) (Reference 1), provided the coordinates of the corners of the ESP PPE (ESP site footprint) based on the North Anna Plant Site coordinate reference system.

Coordinate references based on the State NAD 83 South Zone coordinate system are provided in Figure 1 at the end of this response.

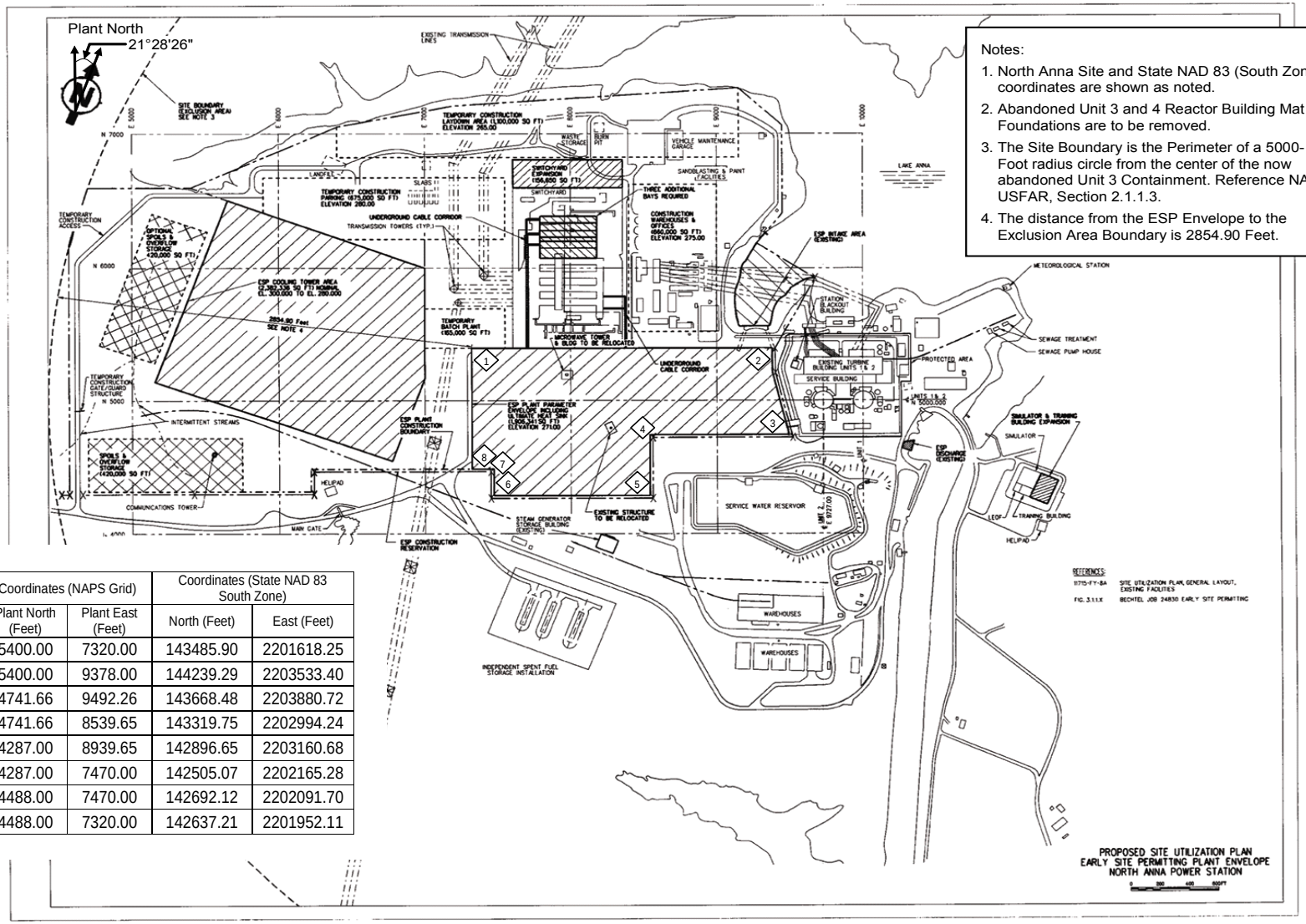
Both coordinate system units are "feet."

Reference

1. August 2, 2004 Letter from Eugene S. Grecheck, Vice President-Nuclear Support Services, Dominion, to U.S. Nuclear Regulatory Commission, Document Control Desk, "Dominion Nuclear North Anna, LLC, North Anna Early Site Permit Application, Response to Request for Additional Information No. 4."

Application Revision

None.



Notes:

1. North Anna Site and State NAD 83 (South Zone) coordinates are shown as noted.
2. Abandoned Unit 3 and 4 Reactor Building Mat Foundations are to be removed.
3. The Site Boundary is the Perimeter of a 5000-Foot radius circle from the center of the now abandoned Unit 3 Containment. Reference NAPS USFAR, Section 2.1.1.3.
4. The distance from the ESP Envelope to the Exclusion Area Boundary is 2854.90 Feet.

	Coordinates (NAPS Grid)		Coordinates (State NAD 83 South Zone)	
	Plant North (Feet)	Plant East (Feet)	North (Feet)	East (Feet)
1	5400.00	7320.00	143485.90	2201618.25
2	5400.00	9378.00	144239.29	2203533.40
3	4741.66	9492.26	143668.48	2203880.72
4	4741.66	8539.65	143319.75	2202994.24
5	4287.00	8939.65	142896.65	2203160.68
6	4287.00	7470.00	142505.07	2202165.28
7	4488.00	7470.00	142692.12	2202091.70
8	4488.00	7320.00	142637.21	2201952.11

DSER Open Item 2.4-2 (DSER page 2-62)

> The applicant provided a figure that contains a layout of the ESP intake and discharge tunnels. Based on SSAR Figure 1.2-4, the staff determined that parts of the ESP intake and discharge tunnels will be located outside the PPE (ESP footprint). The applicant needs to specify minimum distances from the SSCs of the existing units to the ESP intake and discharge tunnels to ensure no interference will occur. This is **Open Item 2.4-2.** Once these distances are provided, and assuming the staff agrees with them, the staff plans to impose these distances as **Permit Condition 2.4-1** to ensure that no such interference will occur if a COL or CP is ultimately granted.

Response

The response to RAI 2.4.1-1 c) (Reference 1) stated that the intake tunnels for a new Unit 3 would be routed from the ESP intake area south a distance of approximately 200 feet to the ESP footprint. The discharge tunnel for Unit 3 would be routed from the ESP footprint east a distance of up to 1800 feet to the ESP discharge. SSAR Figure 1.2-4 shows the locations of the ESP intake area, the ESP footprint, and the ESP discharge. These routings generally coincide with those originally planned for abandoned Units 3 and 4, which were never completed. (Note that a new Unit 4 would use a closed-cycle cooling system with dry cooling towers to transfer rejected heat to the atmosphere.)

As shown on SSAR Figure 1.2-4, new Unit 3 would have its own intake west of the existing units and its own outfall adjacent to the existing units' outfall at the head of the discharge canal. The preliminary construction strategy would be to use existing structures and routes to the extent possible. In the event that the existing tunnels from the abandoned units are deemed unsuitable, new tunnels would be constructed in the same vicinity. While the routing for these tunnels would pass beneath roadways, power lines, fence lines, etc., the tunnels would remain well away from the existing units' major power block structures.

The detailed design for the new unit intake and discharge tunnels has not been established. The design, locations, routing, and construction details would be determined as part of detailed engineering. As part of that effort, the existing portions of the intake and discharge tunnels and associated structures from the abandoned units would be evaluated for suitability.

Dominion has extensive experience evaluating the suitability of proposed construction activities and with the safe implementation of major plant modifications at existing operating sites. This experience includes steam generator replacement, reactor

pressure vessel head replacement, transformer replacement, service water piping replacement, and other major modifications at the North Anna site.

The feasibility of safely performing the construction activities associated with the intake and discharge tunnels for a new Unit 3 has been reviewed. This review included an assessment of the types of construction activities that would be required and whether appropriate compensatory measures could be implemented to safely perform the work including, for example:

- Sheet piling
- Temporary or permanent barriers
- Limits on construction equipment travel paths, operating areas, crane boom swings, etc.
- Shutdown/isolation of systems and components in proximity to the work
- Temporary alternate operating system configurations
- Hand excavation in close proximity to existing permanent plant equipment
- Other typical safe construction practices

That review concluded that it would be feasible to perform the construction activities associated with the intake and discharge tunnels with no adverse interactions on the existing units.

As part of the detailed engineering described above, construction activities for the intake and discharge tunnels for new Unit 3 would be evaluated for potential impacts on the existing units. In addition to the applicable Part 52 requirements, the evaluation would need to satisfy the provisions of the existing units' 10 CFR Part 50 operating licenses, in particular, the requirements of 10 CFR 50.59.

Based on the approach described above, reasonable assurance is provided that no adverse interactions between existing and new units would occur as a result of future construction activities associated with the intake and discharge tunnels for a new Unit 3.

Dominion requests that this DSER Open Item be closed and that no permit condition be imposed.

Application Revision

None.

DSER Open Item 2.4-3 (DSER page 2-62)

> The applicant estimated a margin of 5.9 m^3/s (209 cfs) in the water budget, assuming that the average net inflow of 10.5 m^3/s (370 cfs) would always be available. Nonsafety-related cooling water needs for all units, including the proposed additional units, are 3.4 m^3/s (121 cfs), and a minimum release of 1.1 m^3/s (40 cfs) from Lake Anna is required by the State of Virginia. However, during periods of low flow, the expected inflow into Lake Anna can be substantially lower than the average inflow. These periods may be critical for nonsafety-related cooling needs. The applicant needs to describe the potential impacts of low-flow conditions on the operation of all units. This is **Open Item 2.4-3.**

Response

The water budget analysis that was conducted to assess potential impacts of low-flow conditions on the operation of all units is described in detail in ER Section 5.2.2, which is referenced and summarized in SSAR Section 2.4.11.4. This water budget analysis determined that the minimum Lake Anna water level during the simulation period would have been 242.6 ft msl if new Unit 3 had been operating along with the existing Units 1 and 2. This simulation period included several periods of extended low inflow into Lake Anna including the severe 2001-2002 drought.

SSAR Section 2.4.11.1 states that the existing units can continue to operate with lake levels as low as 244.0 ft msl before shutdown of the units would be required by the existing Units 1 and 2 Technical Requirements Manual. The SSAR further indicates that modifications were underway to reconfigure the intake for Units 1 and 2 and revise the Technical Requirements Manual to allow operation down to 242.0 ft msl. Subsequent to submitting the SSAR, these modifications have been completed. The current minimum lake level for existing Units 1 and 2 is now 242.0 ft msl. As stated in SSAR Section 2.4.11.1, the minimum lake level for operation of new Unit 3 would be 242.0 ft msl.

Because the minimum lake level (242.0 ft msl) required for normal plant operation is less than the minimum lake level determined from the water budget analysis (242.6 ft msl), there would be no impacts on normal plant operation even during extended periods of low net inflow to Lake Anna.

The Technical Requirements Manual would continue to require plant shutdown if the lake level was not adequate to support normal operation. Because lake level changes slowly even during periods of reduced inflow, any shutdown resulting from this requirement would be a planned shutdown.

Application Revision

The 3rd paragraph of SSAR Section 2.4.11.1 will be revised to read as follows:

> Lake Anna, which was formed by the construction of the North Anna Dam on the North Anna River, provides cooling water for the existing units. Lake Anna would also provide cooling water for the new units, as described in Section 2.4.1. Currently, the lake is maintained at an operating water level of 250 ft msl. The existing units can continue to operate with lake water levels as low as Elevation 242.0 ft msl before shutdown of the units must occur in accordance with the plant's Technical Requirements Manual (Reference 33). For the new units, the anticipated minimum lake level for operation is also Elevation 242.0 ft msl. All intake elevations for cooling water and plant service water needs would be based on this elevation, with sufficient margin to ensure plant operation during low water events. The historic low water levels in Lake Anna are presented in Section 2.4.11.3.

The 2nd paragraph of SSAR Section 2.4.11.4 will be revised to read as follows:

> The minimum calculated Lake Anna water levels for the Existing and Proposed scenarios are 245.1 and 242.6 ft msl, respectively. The durations of low lake water levels from the analysis are shown in Table 2.4-6. The minimum operating level for existing Units 1 & 2 and new Unit 3 (Elevation 242.0 ft msl) is below the minimum calculated under the Proposed scenario. Therefore, there would be no new impacts of low-flow conditions on the operation of either the existing Units 1 and 2 or new Unit 3.

Reference 33 of SSAR Section 2.4 References will be revised to read as follows:

> 33. Technical Requirements Manual for North Anna Units 1 & 2, Revision 41, Dominion, March 24, 2004.

ER Section 3.4.1.3.3 will be revised to read as follows:

> The water level in Lake Anna is currently regulated by the North Anna dam to maintain a normal lake level of 250 ft msl to support operation of the existing units. Fluctuations of the inflows to the lake cause the lake level to temporarily go above or below the normal design level of 250 ft msl. According to the existing units' Technical Requirements Manual, 242 ft msl is the minimum lake level for the Unit 1 and 2 circulating water systems to continue operation. With the additional water supply demand from the new units, the water budget analysis in Section 5.2.2 indicates that the lake level will not drop below 242 ft msl during

severe drought conditions. For the future concurrent operation, the normal lake level would be maintained at 250 ft msl.

The 5th paragraph of ER Section 5.2.2.1.3 will be revised to read as follows:

Table 5.2-4 provides the water level duration-frequency for the low water levels of interest to Lake Anna users and the minimum water level for the 24-year simulation period. These results demonstrate that the percent of time that the water level is less than or equal to a given elevation increases with the increasing plant cooling water needs associated with the addition of Unit 3. The results also indicate that the minimum water level for the simulation period decreases with increasing plant cooling water needs of Unit 3. Note that this simulation models the existing units as continuing to operate to a minimum elevation of 242.6 ft msl which is above the minimum elevation specified in their Technical Requirements Manual (Reference 8).

The 2nd to last paragraph of ER Section 5.2.2.2 will be revised to read as follows:

Lake drawdown to Elevation 242.6 ft msl would not impact the existing units. The Technical Requirements Manual for the existing units requires plant shutdown when the lake level drops below Elevation 242 ft msl (Reference 8). Results included in Table 5.2-4 indicate that lake levels would not fall to Elevation 242 ft msl when Unit 3 is added.

Reference 8 of ER Section 5.2 References will be revised to read as follows:

8. Technical Requirements Manual for North Anna Units 1 & 2, Revision 41, Dominion, March 24, 2004.

DSER Open Item 2.4-4 (DSER pages 2-96 and 2-97)

In SSAR Section 2.4.7.3, the applicant discussed historical ice formation in the region. The applicant reported that, after the construction of the dam and before the start of the operation of the existing NAPS units, an ice sheet formed on the lake during the winter of 1977. Since NAPS began operating, ice sheets have formed only on the upper reaches of Lake Anna (upstream of the Route 208 bridge). The staff accessed the USACE historical database of ice jams on August 2, 2004. One ice jam was reported over the past 70 years for the North Anna River, on March 4, 1934, near the Doswell USGS gauge located approximately 25.7 km (16 mi) downstream of the ESP site. This observation suggests that ice jam formation upstream of the ESP site is possible. The breakup of an upstream ice dam may result in flood waves at the ESP site. SSAR Section 2.4.7 does not provide regional characteristics of the location, duration, height of ice dams, and ice-induced high flows.

Because there is an historical record of ice jams on the North Anna River, the staff determined that the applicant should address the possibility of an ice jam or an ice dam formation upstream of the ESP site, and should estimate the effect of a flood wave generated from the breakup of such an ice formation. This is **Open Item 2.4-4**.

Response

To assess the effect of a flood wave generated from the breakup of an ice jam or an ice dam formation upstream of the ESP site, an ice dam was postulated to occur on the North Anna River close to where it enters Lake Anna. Of the various tributaries that enter the lake, the North Anna River was selected because it is the largest tributary to the lake and has the potential to impound the greatest volume of water. The volume of water that could be impounded behind an ice dam at this location and the effect of this postulated ice dam breaching on downstream flooding are described below.

The most common location for an ice jam to form is in an area where the river slope changes from a relatively steep slope to mild slope (Reference 1). Inspection of the USGS topographic maps of the North Anna River (References 2-4) indicate that such a river slope transition occurs at about Elevation 260.0 ft, indicating that this a location where an ice jam could potentially form. (The normal Lake Anna water level elevation is 250.0 ft.) This location was therefore assumed for the purposes of estimating the volume of water that could be impounded by an ice dam.

Descriptions of the ice jams in the USACE database (Reference 5) provide limited data regarding the physical characteristics of the ice jams themselves. Only one record in

the State of Virginia, the December 1989 ice jam on the Rappahannock River at Fredericksburg, Virginia, provides a physical description of the ice flow. The description indicates that the ice jam had a maximum thickness of about 4 to 6 feet. Based on this information, an ice dam height of 10 ft was assumed to estimate the volume of water that might be impounded on the North Anna River.

Based on the location and height of an ice dam as described above, the volume of water that could be impounded behind such an ice dam was estimated using the Orange, Virginia topographic map (Reference 4). The surface area of the impoundment was estimated to be about 150 acres assuming the water would be impounded to an elevation of 270 ft msl. The corresponding volume of the impoundment was estimated by multiplying the surface area of about 150 acres by the average depth of the impoundment, which was conservatively assumed to be the 10 ft height of the ice dam. Using this approach, the volume of water impounded behind a 10 ft high ice dam at this location was determined to be about 1500 acre-ft.

To assess the effects of an instantaneous breach of the hypothesized ice dam on downstream flooding, the 1500 acre-ft of water potentially impounded was compared to the volume of water stored in Lake Louisa and Lake Orange. These two reservoirs are located upstream of Lake Anna as discussed in the response to RAI 2.4.4-1 (Reference 6). Lake Louisa is located on Hickory Creek, a tributary to the North Anna River and Lake Orange is located on Clear Creek, a tributary to Lake Anna. The height of the dam for Lake Louisa is 25 ft and the height of the dam for Lake Orange is 44 ft. The combined volume of the two upstream reservoirs is 7671 acre-ft. The response to RAI 2.4.4-1 indicates that a simultaneous breach of both of these dams coincident with the probable maximum flood (PMF) on Lake Anna would not increase flood levels to significantly affect the ESP site.

Because the volume of water impounded behind a hypothesized ice dam (1500 acre-ft) is significantly less than that impounded behind Lake Louisa and Lake Orange (7671 acre-ft), the effects of the ice dam breach are bounded by those associated with the breaching of the man-made impoundments. Additionally, the Lake Anna PMF is based on the maximum all-season PMP, which occurs during warm temperatures and could not occur simultaneous with an ice jam formation. Thus, the impacts due to the breakup of an ice jam on a tributary upstream of the ESP site would not significantly impact the flooding potential of the site because the previously analyzed flooding scenarios are more conservative.

<u>References</u>

1. U. S. Army Corps of Engineers, Engineering Manual EM-1110-2-1612, “Ice Engineering,” Washington D.C., October 30, 2002.

2. U. S. Geological Survey, Mineral Quadrangle, Virginia, 7.5 Minute Series (Topographic), 1981.

3. U. S. Geological Survey, Lahore Quadrangle, Virginia, 7.5 Minute Series (Topographic), 1981.

4. U. S. Geological Survey, Orange Quadrangle, Virginia, 7.5 Minute Series (Topographic), 1994.

5. U. S. Army Corps of Engineers, Cold Regions Research and Engineering Laboratory, Ice Jam Data Base, Accessed on line at http://www.crrel.usace.army.mil/ierd/icejam/icejam.htm, February 22, 2005.

6. August 2, 2004 Letter from Eugene S. Grecheck, Vice President-Nuclear Support Services, Dominion, to U.S. Nuclear Regulatory Commission, Document Control Desk, "Dominion Nuclear North Anna, LLC, North Anna Early Site Permit Application, Response to Request for Additional Information No. 4."

Application Revision

None.

DSER Open Item 2.4-5 (DSER pages 2-98 and 2-99)

The maximum accumulated degree-days below freezing during the period of December 1, 1976, to March 31, 1977, were 178.8 °C (321.8 °F), as shown in Figure 2.4.7-1. The staff used Assur's method to estimate a maximum ice thickness of 43.4 cm (17.1 in.). The staff's estimate is higher than the applicant's estimate of 34.3 cm (13.5 in). However, this difference does not have any safety impact because, as explained below, the increase in ice thickness does not affect the intake for the proposed additional units. The staff intends to include a site characteristic value regarding intake water temperature as discussed in the following paragraph. The ice sheet could be in place for several weeks. The staff determined, based on Figure 3.4-4 of the applicant's Environmental Report and the applicant's commitment to a minimum water level of 73.8 m (242 ft) MSL, that the intake structure for the proposed additional units is at least 6.4 m (20 ft) below the minimum allowable low water level. The staff therefore concluded that the staff-calculated maximum estimated ice thickness of 43.1 cm (17.1 in) would not hamper operation of the proposed additional units. However, the staff also determined that extended periods of water temperatures at freezing are possible near the intake structure.

In response to RAI 2.4.7-2, the applicant stated that formation of frazil and anchor ice is an extremely rare condition that can only happen when all units are shut down and prolonged, wintry conditions prevail. The applicant stated that this issue would be addressed during design of the intake structures. However, the staff has determined that minimum lake temperature is a site characteristic important as a design basis for a nuclear power plant that might be constructed on the site, and therefore this is **Open Item 2.4-5.** The staff intends to include this as a site characteristic value in any ESP that the NRC may issue for this ESP application.

<u>Response</u>

The only scenario in which frazil and anchor ice could form is one in which all units have been shutdown for a prolonged period in the winter, allowing the lake cool to ambient temperatures, and conditions conducive to frazil ice formation are present. These conditions are associated with open water (no surface ice), freezing air and water temperatures, strong winds, and clear nights. SSAR Section 2.4.7.4 describes this scenario, and indicates that no safety-related facilities would be impacted if frazil ice were to form in the intake area.

The response to RAI 2.4.7-2 indicates that the possibility of anchor ice accumulating on the trash racks and screens of the intake structure is remote and would be assessed

during detailed engineering. The response also indicates that measures would be included in the design of the intake structure to preclude the formation of anchor ice if the assessment concluded that anchor ice could form. Such measures might include, but not be limited to, heating intake components subject to anchor ice accumulation, re-circulating warm water to the intake, and using coatings that reduce the ice adhesion strength.

In lieu of a minimum lake temperature, the potential for frazil and anchor ice formation will be identified as a site characteristic for the cooling water intake structure.

Application Revision

SSAR Table 1.9-1 will be revised to add the following new site characteristic:

Item	Single Unit/Group Value [Second Unit/Group Value]	Description and References
Cooling Water Intake Structure Ice Formation	Potential for formation of frazil and anchor ice [Same for 2nd unit/group]	▪ Refer to Section 2.4.7.4

DSER Open Item 2.4-6 (DSER page 2-102)

Section 2.4.3 of this SER presents the staff's evaluation of the ability of Lake Anna (including the W HTF) to survive a PMF. The staff did not consider Lake Anna a safety-related reservoir, since it is not a part of the proposed UHS for the proposed units.

The applicant stated that the UHS for the proposed additional units would consist of a mechanical draft cooling tower over a buried water storage basin. This UHS would have its own source of water that would be independent of the lake.

The applicant suggested that the proposed Unit 3 would use a once-through cooling system during normal plant operation. The applicant also suggested that the proposed Unit 4 would use a closed-cycle cooling system with dry towers during normal plant operation. The limitation on the quantity of cooling water and other attributes of the cooling system design for the proposed Units 3 and 4 are site constraints. Consequently, the staff intends to identify these items as site characteristics in any ESP the NRC might issue for the proposed ESP site.

The applicant did not provide details of the location and construction of the UHS buried water storage basin. These details are needed because they relate to the reliability and stability of the UHS under the pressure head of ground water, which is at the grade level at certain locations of the ESP site. Therefore, the staff could not review these details. These data are needed and are part of RAIs 2.4.1-1 and 2.4.4-2. The need for location and construction details to determine differential head between groundwater and the UHS is **Open Item 2.4-6.**

<u>Response</u>

Details of the location and construction of the UHS have not been established. If the chosen reactor design requires a conventional UHS, the design, location, and construction details of the UHS would be determined as part of detailed engineering and described in the COL application.

The response to RAI 2.4.4-2 (Reference 1) describes that the estimated inside dimensions of each UHS cooling tower basin would be approximately 235 ft wide by 350 ft long by 50 ft deep. These dimensions are based on a bounding storage volume of 30,600,000 gallons from SSAR Table 1.3-1. Assuming, for preliminary estimates, that the basin walls are 2.5 ft thick, and the base mat is 5 ft thick, the outside dimensions would be about 240 ft wide by 355 ft long by 55 ft deep.

The extreme design condition for hydrostatic uplift of the basin would be with groundwater at the surface and no water in the basin, i.e., a differential head of 55 ft. This would produce a hydrostatic uplift pressure on the bottom of the basin of about 3.4 ksf. The UHS would be designed to resist this uplift pressure, including an appropriate factor of safety. The uplift pressure would be resisted by:

- The weight of the UHS structure.
- The shear resistance of the backfill. At least half of the 55 ft depth of the basin could be expected to be in competent bedrock. If the excavation in the rock were backfilled with lean concrete, this would provide a significant amount of uplift resistance. Installing shear keys on the external wall of the basin could further increase this resistance.
- Rock anchors. The foundation of the basin would be in competent bedrock. Rock anchors drilled into this bedrock would provide, if needed, the balance of the uplift resistance not supplied by the weight of the structure and the shear resistance of the backfill. The anchor contribution could be increased as required by increasing the length and diameter of the anchors, and/or reducing their spacing.

In order to assess the feasibility of UHS design to resist the potential for hydrostatic uplift, the following provides a description of the typical design considerations that would be evaluated as part of detailed engineering.

1. Hydrostatic Uplift on Base of UHS Basin

   Based on the outside dimensions of the basin given above, the base area of the basin is:

   240 ft x 355 ft = 85,200 ft^2

   With groundwater level at the ground surface, the hydrostatic uplift pressure on the base of the basin is:

   55 ft x 62.4 pcf = 3,432 psf = 3.432 ksf.

   Thus, total hydrostatic uplift force on the base of the basin is:

   3.432 ksf x 85,200 ft^2 = 292,406 kips.

2. Contribution of Basin Weight to Uplift Resistance

The volume of concrete in the basin, based on the basin dimensions given above is 573,500 ft^3 ($\approx$ 21,250 yd^3).

Assuming the unit weight of concrete = 150 pcf = 0.15 kcf, the weight of concrete in the basin = 0.15 x 573,500 = 86,025 kips.

Applying a factor of safety of 1.25 to the weight, the allowable uplift resistance due to basin weight = 86,025/1.25 = 68,820 kips. Note that this conservatively neglects the weight of any aboveground components.

Uplift force that must be resisted by backfill friction/adhesion and/or rock anchors: 292,406 – 68,820 = 223,586 kips.

3. Contribution of Backfill Friction/Adhesion to Uplift Resistance

As noted above, approximately the bottom half of the 55-ft deep excavation for the basin would be expected to be in competent material (Zone III-IV or Zone IV rock). If lean concrete backfill is poured between the excavated rock face and the basin wall, a significant amount of uplift resistance would be developed.

According to Equation 11.15 of Reference 2, the ultimate side resistance, f_s, between a concrete drilled shaft in rock and the rock is:

$f_s = 2.5(q_u)^{0.5}$ for $q_u > 280$ psi

where q_u is the unconfined compressive strength of the rock or concrete in psi, whichever is less. This equation can be used to compute f_s between the lean concrete backfill and the basin wall, and between the lean concrete backfill and the excavated rock face. According to SSAR Table 2.5-45, the design strength of the Zone III-IV rock is 4,000 psi. Assuming that the design strength of the concrete basin is 4,000 psi and the strength of the lean concrete backfill is 2,500 psi:

q_u = 2,500 psi
and $f_s = 2.5(2{,}500)^{0.5}$ = 125 psi = 18 ksf.

Using a factor of safety of 2.5 on this ultimate skin friction/adhesion would result in the allowable skin friction/adhesion of the lean fill against the concrete basin or against the rock excavation of 7.2 ksf. (18/2.5=7.2).

The critical contact would be between the concrete backfill and the concrete basin, since the perimeter of the basin is less than the perimeter of the excavated rock surface.

The circumference of the basin walls = 2 x (355 + 240) = 1190 ft.
Allowable uplift resistance per foot height of wall = 7.2 x 1190 = 8,568 kips.

Thus, to produce 223,586 kips of uplift resistance (i.e., total hydrostatic uplift force minus allowable basin weight), the height, H, of lean concrete backfill needed is:

H = 223,586/8,568 = 26.1 ft

This value of approximately 26 ft height of concrete backfill required to resist the balance of the uplift resistance neglects the contribution made by granular backfill that will be placed above the lean concrete. Computations show that a compacted granular backfill above the lean concrete would add about 5,000 kips to the allowable uplift resistance.

The above example computation demonstrates that the weight of the basin plus the contribution friction/adhesion of about 26 ft of lean concrete backfill to uplift resistance would be sufficient to prevent flotation of the empty basin.

4. Contribution of Rock Anchors to Uplift Resistance

Rock anchors beneath the structure could be used as an alternative or supplement to concrete backfill. The rock below the UHS basin would almost certainly be Zone IV material with a design unconfined compressive strength of 12,000 psi (SSAR Table 2.5-45). To construct the anchors in this rock, small diameter (6 to 12 in.) holes would be drilled about 20 ft deep into the rock, high strength steel threaded bars would be centered in the holes, and high strength grout pumped into the holes. There would be sufficient stick-up of the bar to develop the capacity of the rock anchor in the 5-ft thick basin foundation mat.

The capacity of the rock anchor would be the lesser of (a) the strength of the steel threaded bar, and (b) the adhesion between the pumped grout and the drilled rock.

(a) Assume a DSI (Dywidag-Systems International) nominal 1¾-in. diameter high-strength threaded anchor is used (actual diameter is 1.83 in.) with an ultimate tensile strength of 150 ksi, giving an ultimate strength of about 400 kips. Typically, the rock anchors are designed so that a test load of

150% of the design load may be applied without exceeding 80% of the ultimate anchor tensile strength. Thus, the design load would be 213 kips. Note that the bars would need permanent corrosion protection.

(b) Assume that the anchor grout has a strength of 5,000 psi. From Equation 11.15 of Reference 2:

$f_s = 2.5(5{,}000)^{0.5} = 176.8$ psi = 25.5 ksf, rounded to 25 ksf in this example.

Assume the 1¾-in. diameter high-strength threaded DSI anchor has a length of 24 ft. Assume that there is 4 ft stick-up for development in the base mat. Length into rock is 20 ft. Assume 6-in. diameter hole (0.5 ft diameter).

Ultimate grout-rock bond strength = π x 0.5 x 20 x 25 = 785 kips.

Thus, for a 213-kip anchor, there is a factor of safety of 785/213 = 3.69 against bond failure between the grout and the rock. This would be more than sufficient.

Note that 20-ft development length between the bar and the grout in the anchor would be several times longer than required. The 4 ft development length into the base mat would be sufficient. If not, extra measures such as putting large nuts on the bars would make it sufficient.

Number of rock anchors required to hold down 223,586 kips of uplift resistance = 223,586/213 = 1050.

Area per rock anchor = mat area/no. of rock anchors = (240 x 355)/1050 = 81.1 ft^2

Center-to-center spacing of rock anchors = $(81.1)^{0.5}$ = 9.0 ft.

Note that this number of rock anchors assumes that there is no backfill resistance. Even if lean concrete were not used, there would be granular backfill between the basin and excavation walls. Computations show that 55-ft depth of compacted granular backfill would add over 15,000 kips to the allowable uplift resistance.

5. Summary

- The bounding storage volume would require an UHS basin with inside dimensions of about 235 ft wide by 350 ft long by 50 ft deep. Assuming 2.5-ft-thick walls and a 5-ft-thick base mat, the outside dimensions of the basin would be 240 ft wide by 355 ft long by 55 ft deep. Actual dimensions would depend on the reactor design chosen and would be determined as part of detailed engineering.

- If the UHS basin was empty and the groundwater level was at the ground surface, the hydrostatic uplift force on the base of the basin would be about 292,400 kips.

- The weight of the UHS basin below ground surface that could be used to resist the hydrostatic uplift would be about 68,800 kips, neglecting the weight of aboveground components. This number includes a safety factor of 1.25.

- Assuming at least 26 ft depth of competent rock at the bottom of the excavation, backfilling these 26 ft with 2,500 psi lean concrete would provide the balance of the uplift resistance (about 223,600 kips). This includes a factor of safety of 2.5. An additional 5,000 kips allowable uplift resistance would be obtained from the granular backfill above the lean concrete.

- If granular backfill were used instead of lean concrete, short high-strength rock anchors could be used as an alternative to provide the balance of the uplift resistance. Approximately 1050 of these rock anchors would be needed, spaced at 9-ft centers. The rock anchors would be 20 ft long, 6 in. diameter, with a 1¾-in. diameter high-strength steel threaded bar, and 5,000 psi grout. The steel rock anchors would need permanent corrosion protection. The estimated number of rock anchors conservatively neglects the more than 15,000 kips uplift resistance that would be provided by granular backfill around the basin.

On page 2-83 of the DSER, the NRC has identified Permit Condition 2.4-5, which would impose a condition that “the free surface elevation of the UHS may not fall below 82.3 m (270 ft) MSL.” The DSER explains that this permit condition is needed for any uplift of a UHS basin caused by buoyancy. Based on the preceding description of the typical engineering practices that would be evaluated as part of detailed engineering to resist the potential for hydrostatic uplift, no permit condition on the free surface elevation of the UHS would be necessary.

References

1. August 2, 2004 Letter from Eugene S. Grecheck, Vice President-Nuclear Support Services, Dominion, to U.S. Nuclear Regulatory Commission, Document Control Desk, "Dominion Nuclear North Anna, LLC, North Anna Early Site Permit Application, Response to Request for Additional Information No. 4."

2. U. S. Department of Transportation, Federal Highway Administration. "Drilled Shafts," Publication N0. FHWA-HI-88-042, July 1988.

Application Revision

None.

DSER Open Item 2.4-7 (DSER page 2-116)

> Observed increases in water levels in the new wells ranged from less than 0.3 m (1 ft) to more than 1 m (3 ft) over the period of December 17, 2002, through June 17, 2003. The applicant included previously existing wells monitored at the same time in the analysis. The observed variation in water levels in wells could be significant, but represented only a 6-month period. The staff evaluated additional information the applicant provided in response to RAI 2.4.12-1, but found that it needed additional data to determine whether the new ground water level measurements correlate with data from the long-term piezometers. Groundwater measurements should contain at least one full year of data to determine recent seasonal fluctuation in ground water levels at the ESP site. This is **Open Item 2.4-7.**

Response

At the time the groundwater level measurements for the SSAR were initiated at the ESP site, the area had been going through a period of severe drought. This drought period ended in Fall 2002 just as the groundwater level measurements were beginning. A total of 4 rounds of quarterly measurements were obtained in the period beginning December 17, 2002 and ending September 29, 2003 as reported in SSAR Table 2.4-15. To address this open item, an evaluation has been performed to determine if the 4 quarterly measurements reflect the fully recovered condition of the groundwater table at the end of the drought period. If the previous measurements do not reflect the recovered condition, the horizontal hydraulic could be underestimated.

A supplementary round of groundwater level measurements were taken in the wells on February 1, 2005, about 1 year 4 months after the last quarterly groundwater level measurement and over 2 years after the drought had ended. Sufficient time has elapsed to allow the groundwater table to recover from the 2002 drought. SSAR Table 2.4-15 and SSAR Figure 2.4-15 have been updated to include the February 1, 2005 measurements (provided at the end of this response). In addition, Figures 1, 2, and 3 from the response to RAI 2.4.12-1 (Reference 1) have been updated with the additional groundwater level measurements (provided at the end of this response). The wells installed specifically for the ESP investigation generally show steady to slightly falling water levels between the current measurements and the previous measurements taken on September 29, 2003. Two wells (OW-842 and OW-847), however, show an increase in the groundwater level over this time period of 2.2 and 2.5 feet, respectively. Water levels in the previously existing site wells during this time period exhibit changes ranging from –0.2 to +1.7 ft with the majority of changes at 1 ft or less. Note that the water levels observed in the pre-existing piezometers P-10, P-14, and P-18 have been added to the revised SSAR Figure 2.4-15. For the period beginning December 17,

2002 and ending February 1, 2005, the levels from the pre-existing piezometers generally behave like the levels in the wells installed to support the ESP investigation.

The horizontal hydraulic gradient from the center of the ESP site footprint to Lake Anna has been recalculated using the February 1, 2005 measurements. The resulting value of 0.029 ft/ft is bounded by the gradient of 0.03 ft/ft provided in SSAR Section 2.4.12.1.2. Therefore, the current hydraulic gradient reported in SSAR Section 2.4.12.1.2 was not underestimated as a consequence of the 2002 drought. See also the response to DSER Open Item 2.4-10.

Reference

1. August 2, 2004 Letter from Eugene S. Grecheck, Vice President-Nuclear Support Services, Dominion, to U.S. Nuclear Regulatory Commission, Document Control Desk, "Dominion Nuclear North Anna, LLC, North Anna Early Site Permit Application, Response to Request for Additional Information No. 4."

Application Revision

The 5th paragraph of SSAR Section 2.4.12.1.2 will be revised to read as follows:

> Groundwater at the ESP site occurs in unconfined conditions in both the saprolite and underlying bedrock. The results of previous investigations at the site indicate that a hydrologic connection exists between the saprolite and the bedrock. (Reference 45) This condition has been confirmed as part of the ESP subsurface investigation program (Appendix 2.5.4 B) by the presence of nearly equal water level elevations recorded in two observation wells (OW-845 and OW-846, Table 2.4-15) installed adjacent to each other and sealed in the bedrock and saprolite, respectively. At the ESP site, the water table is considered to be a subdued reflection of the ground surface and, therefore, the direction of groundwater movement is toward areas of lower elevations (Reference 45). Measurements made on a quarterly basis between December 2002 and September 2003 and again in February 2005 in observation wells at the site exhibit water level elevations ranging from about Elevation 241 ft msl to Elevation 314 ft msl, with corresponding ground surface elevations of about Elevation 283 and Elevation 335 ft msl, respectively (Table 2.4-15). The measurements shown in Table 2.4-15 represent four quarterly rounds of groundwater level measurements and a supplementary measurement taken at the ESP site to characterize seasonal variability in the water levels. Figure 2.4-15 presents hydrographs based on the water levels provided in this table for the nine observation wells (OW-841 through OW-849) installed during the ESP subsurface investigation program and three existing long-term site monitoring

wells (P-10, P-14 and P-18). The other wells that were monitored (P- and WP-) were installed previously for NAPS groundwater monitoring purposes around the SWR and the ISFSI, respectively.

The 5th paragraph of SSAR Section 2.4.12.3 will be revised to read as follows:

Because the existing units' groundwater monitoring wells were not considered to be of sufficient areal extent to determine groundwater levels beneath the ESP site, 9 additional observation wells were installed as part of the ESP subsurface investigation program. Water levels in these 9 wells and 10 of the existing units' monitoring wells were measured quarterly for one year, followed by a supplementary measurement in February 2005, to provide data on groundwater flow direction, gradient, and seasonal groundwater level fluctuations at the site.

The 2nd paragraph of SSAR Section 2.4.12.4 will be revised to read as follows:

One groundwater observation well (OW-844) was constructed at the existing plant grade as part of the ESP subsurface investigation program (Appendix 2.5.4 B). The well is located near the toe of the slope north of the SWR (Figure 2.4-16). A second well (OW-841) was constructed in the partially backfilled excavation for abandoned Units 3 and 4. The top of this well is about 20 feet below the plant grade. Maximum measured groundwater level elevations in these wells ranged from about Elevation 250 feet in OW-841 to Elevation 267 feet in OW-844 between December 2002 and February 2005 (Table 2.4-15). Considering the general conformance of the location of OW-844 with the water table profile presented above, these groundwater levels and the piezometric head contours shown on Figure 2.4-16 support the design groundwater level determined for the existing units as described above.

SSAR Table 2.4-15 and Figure 2.4-15 will be replaced with the revised versions shown on the next 3 pages.

Table 2.4-15 Quarterly Groundwater Level Elevations

Observation Well No.	Well Depth* (ft)	Reference Point Elev. (ft)	Reference Point Stickup** (ft)	Top of Well Screen Elev. (ft)	Well Screen Length (ft)	Groundwater Level Elevations Date of Measurement				
						12/17/02	03/17/03	06/17/03	09/29/03	02/01/05
OW-841	34.3	251.6	1.5	228.1	9.7	248.9	249.6	249.6	249.3	249.1
OW-842	49.6	336.7	1.5	297.8	9.6	307.5	308.9	310.8	312.0	314.2
OW-843	49.2	320.6	1.5	282.1	9.7	285.1	288.1	290.8	290.2	290.7
OW-844	24.6	273.5	1.5	257.6	9.6	265.5	266.7	267.3	266.4	266.2
OW-845	55.0	297.3	1.5	253.0	9.7	272.7	274.9	277.4	277.3	277.1
OW-846	32.7	297.3	1.5	273.5	9.8	272.5	274.8	277.1	277.0	276.8
OW-847	49.8	319.7	1.5	280.6	9.6	285.4	287.0	289.5	290.8	293.3
OW-848	47.3	284.5	1.5	240.8	5.0	241.7	242.9	243.6	244.0	243.2
OW-849	49.8	298.5	1.5	259.4	9.7	265.5	269.5	271.7	270.8	269.5
P-10	22.5	286.4	2.4	267.0	5	274.4	274.8	275.2	275.2	275.3
P-14	N/A	327.1	N/A	N/A	N/A	271.6	272.2	272.8	273.1	273.8
P-18	N/A	329.0	N/A	N/A	N/A	285.7	286.5	287.5	288.4	289.9
P-19	58.5	322.3	N/A	N/A	5	284.3	285.2	286.3	287.3	288.9

Table 2.4-15 Quarterly Groundwater Level Elevations

Observation Well No.	Well Depth* (ft)	Reference Point Elev. (ft)	Reference Point Stickup** (ft)	Top of Well Screen Elev. (ft)	Well Screen Length (ft)	Groundwater Level Elevations Date of Measurement				
						12/17/02	03/17/03	06/17/03	09/29/03	02/01/05
P-20	61.0	320.6	N/A	N/A	5	274.9	275.4	275.8	275.0	276.7
P-21	58.5	319.2	N/A	N/A	5	Dry	261.2	262.0	262.4	263.4
P-22	60.0	320.5	N/A	N/A	5	276.8	277.8	278.6	278.9	279.5
P-23	41.2	296.4	1.9	258.7	5	261.1	262.6	263.3	263.1	263.5
P-24	25.0	293.4	2.3	271.3	5	276.4	277.1	278.4	278.3	278.4
WP-3	N/A	317.9(?)	N/A	266.5	5	299.7	301.0	302.8	302.3	302.1
Lake Anna Water Level Elevation						248.1	250.1	250.4	250.1	250.1
Service Water Reservoir Water Level Elevation						314.6	313.3	314.6	314.6	314.5

OW- wells installed in December 2002 as part of ESP Subsurface Investigation Program
P- wells installed previously to monitor NAPS Units 1 and 2 Service Water Reservoir
WP- well installed previously as part of Interim Spent Fuel Storage Installation monitoring program
Below ground surface at time of installation
** Above ground surface at time of installation
N/A - not available

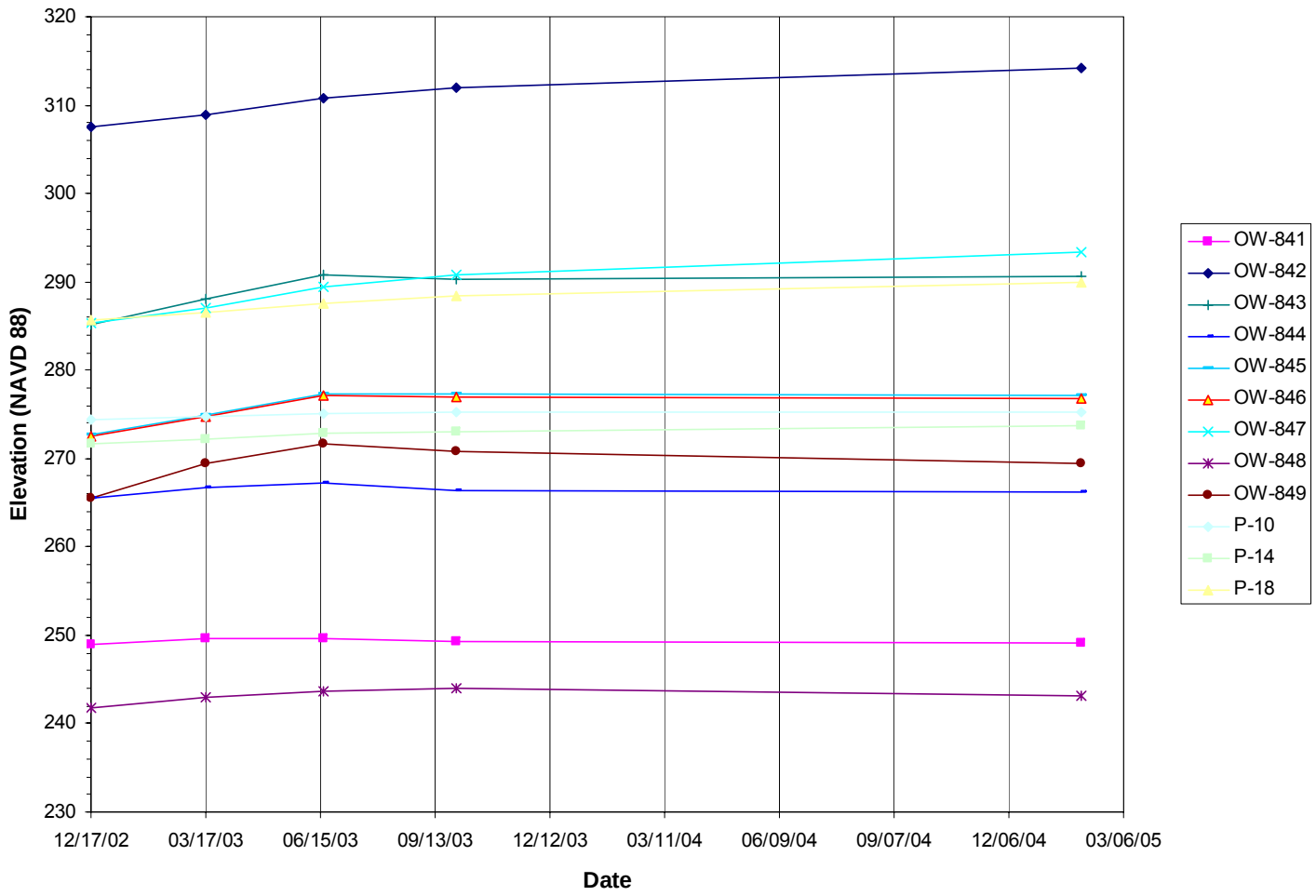


Figure 2.4-15 Groundwater Level Hydrographs

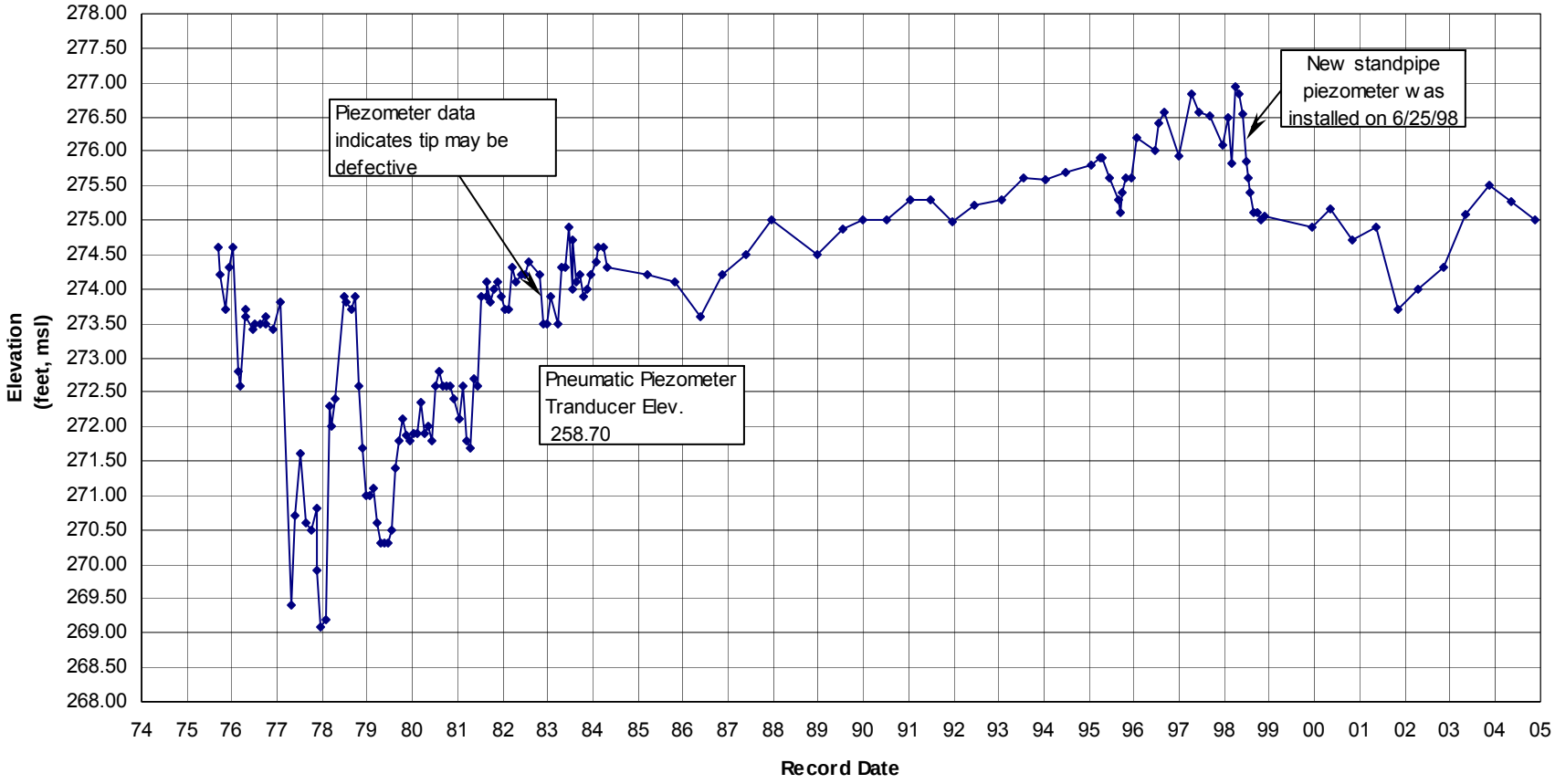


Figure 1. SWR Piezometer P-10

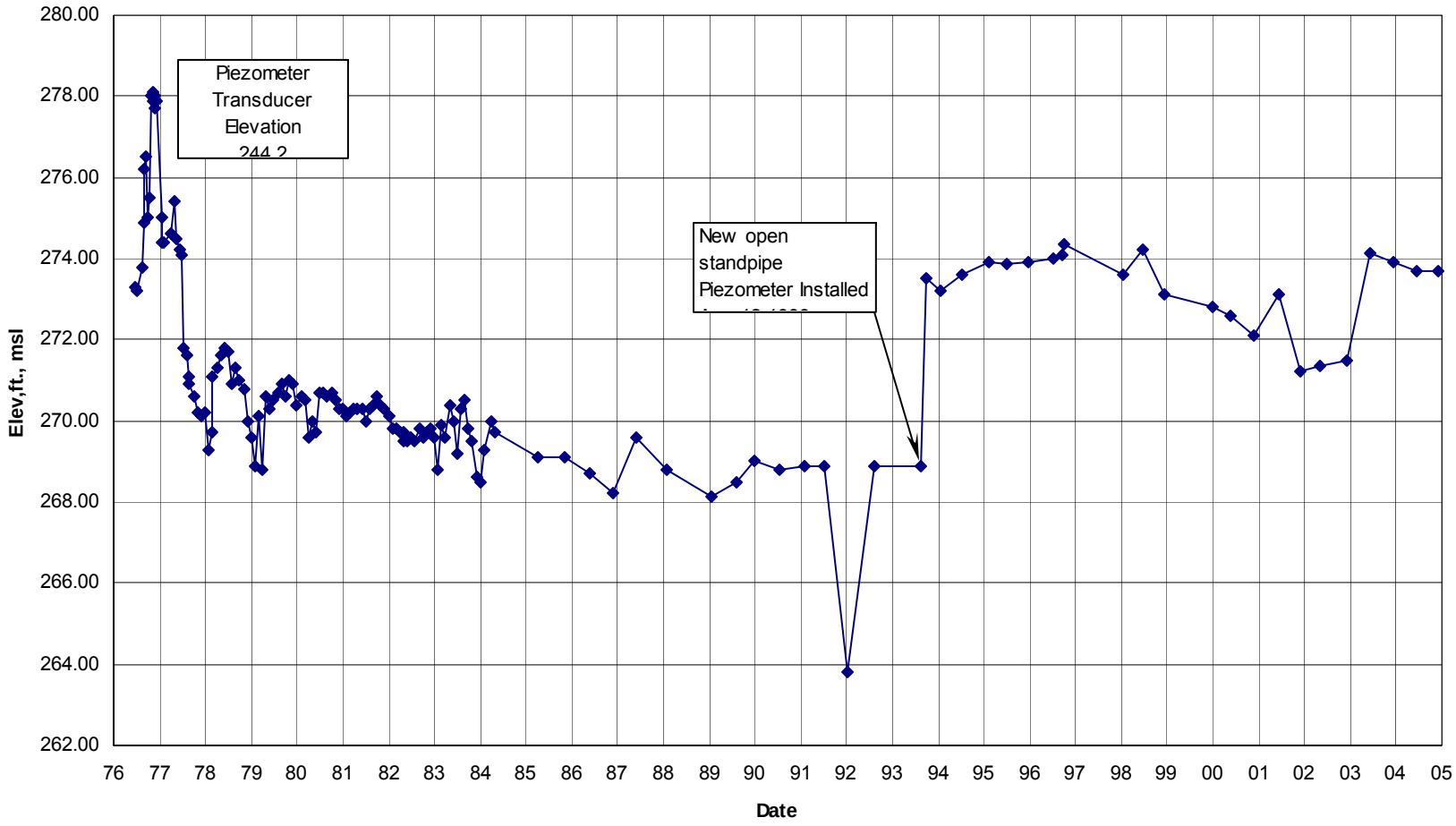


Figure 2. SWR Piezometer P-14

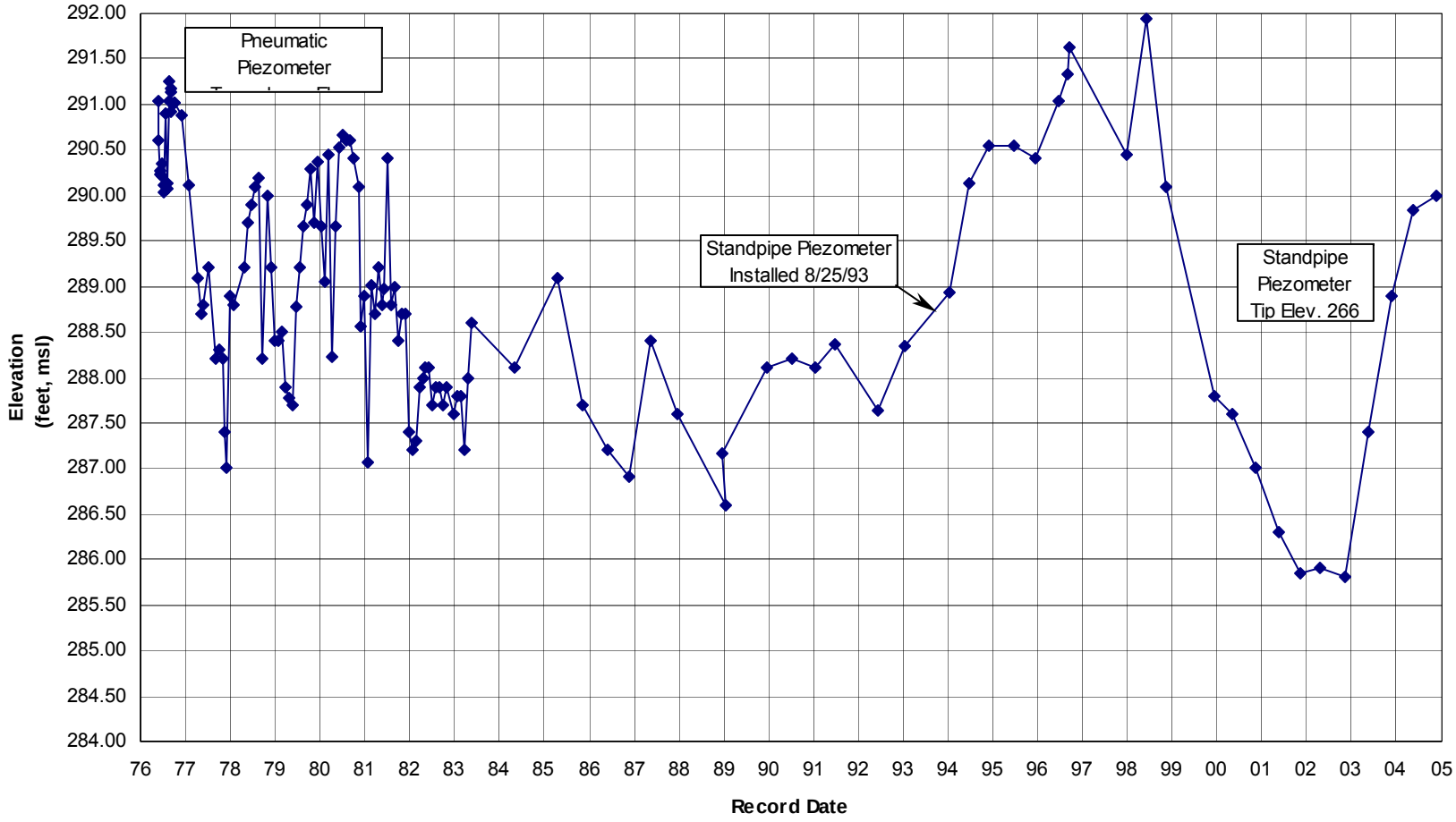


Figure 3. SWR Piezometer P-18

DSER Open Item 2.4-8 (DSER page 2-116)

Ground water discharge to streams and to Lake Anna is significant, ranging from 32 to 67 percent of streamflow, according to several studies conducted in the province cited by Trapp and Horn. The staff was unable to independently estimate the ground water flowpath from the powerblock of the proposed additional units to Lake Anna, since the applicant did not provide the precise location of the powerblock. In the following assessment, the staff used applicant-provided values for effective porosity and distance from the powerblock to Lake Anna.

The staff used the following relationship to determine average ground water velocity:

Velocity = Hydraulic Gradient x Saturated Hydraulic Conductivity/Effective Porosity

The applicant used the geometric mean of the measured hydraulic conductivity values (0.4 m/d(1.3 ft/d)). Use of the geometric mean is not conservative because it results in slower ground water velocity and increased travel time to the environment. Using 1.0 m/d (3.4 ft/d) as the conservative value for hydraulic conductivity, 0.03 m/m (3 ft/100 ft) as the hydraulic gradient, and 0.33 as the effective porosity, the staff estimated the ground water velocity to be 0.09 m/d (0.31 ft/d), as opposed to 0.04 m/d (0.12 ft/d) as reported by the applicant. The staff's calculated travel time from the powerblock to the lake, using 548.6 m (1800 ft) as the distance to the environment, is approximately 16 years, as opposed to the applicant's estimate of 40 years. The applicant needs to explain why a more conservative hydraulic conductivity was not used. This is **Open Item 2.4-8.** The staff intends to identify hydraulic conductivity as a site characteristic in any ESP that might be issued for this application.

Response

Nine groundwater observation wells were installed at the site during the ESP subsurface investigation program. Slug tests were performed in the eight wells open to the saprolite to determine the hydraulic conductivity of this material. The results are presented in SSAR Table 2.4-16. Hydraulic conductivities calculated for the saprolite range from about 0.2 to 3.4 ft/day with a geometric mean value of 1.3 ft/day.

The geometric mean value of 1.3 ft/day presented in the SSAR is representative of the average hydraulic conductivity of the saprolite in accordance with generally accepted industry practice. However, for the purpose of assessing a postulated accidental

release of liquid effluent to the groundwater in a future COL application, it would be appropriate to establish a conservative value for hydraulic conductivity rather than an average value. Consequently, the maximum observed hydraulic conductivity value of 3.4 ft/day will be identified as a site characteristic in SSAR Table 1.9-1.

Application Revision

SSAR Table 1.9-1 will be revised to add the following new site characteristic:

Item	Single Unit/Group Value [Second Unit/Group Value]	Description and References
Maximum Hydraulic Conductivity	3.4 ft/day [Same for 2nd unit/group]	▪ Hydraulic conductivity used to assess the accidental release of liquid effluent to the groundwater. ▪ Refer to Section 2.4.12.1.2

DSER Open Item 2.4-9 (DSER pages 2-120 and 2-121)

SSAR Section 2.4.13 does not contain an analysis of accidental releases to ground and surface waters, which the staff needs to evaluate currently applicable hydrological accidental radionuclide release pathways. The applicant should provide a conceptual model of the subsurface environment, with reference to drill logs, as-built fill, and compaction plans. The subsurface conceptual model should provide estimates, and the basis for these estimates, for the hydraulic conductivity of the soil, surface recharge rates, soil and ambient ground water chemical properties, and piezometric boundary conditions. These model attributes are necessary for the staff to conduct a site-suitability evaluation in accordance with RG 1.113. The staff requested this information in RAI 2.4.13-1.

In its response, the applicant provided details of the hydrogologic characteristics at the ESP site, including a conceptual model of ground water movement through the saprolite and the bedrock underlying the ESP site.

The applicant reported that the only observation of piezometric head difference made between the saprolite and the bedrock indicated an upward hydraulic gradient. The staff needs to understand the implications of an upward hydraulic gradient, with respect to the transport of effluents to the environment. The applicant therefore needs to provide more details about the magnitude, frequency, and spatial location of these upward hydraulic gradients at the ESP site. This is **Open Item 2.4-9.** The staff intends to identify upward hydraulic gradient as a site characteristic in any ESP that might be issued for this application.

Response

Nine observation wells were installed at the site during the ESP subsurface investigation program. Two of these wells, OW-845 and OW-846, were installed as a well pair. OW-845 was installed in bedrock and OW-846 was installed in the saprolite. Groundwater level measurements taken in these wells show the groundwater levels to be nearly equal with a slight upward hydraulic gradient between the bedrock and the saprolite. The groundwater level elevations are shown on SSAR Table 2.4-15 (at the end of the end of the response to DSER Open Item 2.4-7), which has been updated to include a fifth round of water level measurements taken on February 1, 2005.

To address the significance of the upward hydraulic gradient between the bedrock and the saprolite with respect to the transport of postulated effluents in the environment, horizontal and vertical seepage velocities were calculated and compared. The seepage velocities (v) are calculated using the following formula from Reference 1 as

$v = Ki / n_e$ (Equation 1)

where: K = hydraulic conductivity; i = hydraulic gradient; and n_e = effective porosity.

Horizontal Seepage Velocity

The horizontal seepage velocity (v_h) is calculated using Equation 1 and the following input parameter values established in the SSAR: horizontal hydraulic conductivity (K_h) = 1.3 ft/day; horizontal hydraulic gradient (i_h) = 0.03 ft/ft; and effective porosity of 0.33. This yields

v_h = (1.3 ft/day * 0.03 ft/ft) / 0.33
v_h = 0.12 ft/day

Vertical Seepage Velocity

The vertical seepage velocity (v_v) is calculated using Equation 1 and the following input parameters: vertical hydraulic conductivity (K_v); vertical hydraulic gradient (i_v); and the effective porosity. The vertical hydraulic conductivity of the saprolite is calculated assuming a horizontal to vertical anisotropy (Kh:Kv) of 3:1 (Reference 2). Using the geometric mean value of 1.3 ft/day for the horizontal hydraulic conductivity and a vertical anisotropy of 3, the vertical hydraulic conductivity is 0.43 ft/day.

The vertical hydraulic gradient is calculated by dividing the piezometric head difference (Δh) between the two wells (OW-845 and OW-846) by the vertical distance between them (Δz). The head difference between the bedrock and overlying saprolite is calculated using the groundwater level elevations provided in the updated version of SSAR Table 2.4-15 (see the response to DSER Open Item 2.4-7 contained in this letter). The vertical distance between the two wells is taken as the distance between the midpoints of the two well screens and is about 20.5 ft

Using Equation 1, a vertical hydraulic conductivity of 0.43 ft/day, and an effective porosity of 0.33, the vertical seepage velocities calculated for each of the five groundwater level measurements are summarized in the following table:

Item	Groundwater Monitoring Dates				
	12/17/02	**3/17/03**	**6/17/03**	**9/29/03**	**2/1/05**
Head difference, Δh (ft)	0.2	0.1	0.3	0.3	0.3
Vertical distance, Δz (ft)	20.5	20.5	20.5	20.5	20.5
Vertical hydraulic gradient, i_v (ft/ft)	0.01	0.005	0.015	0.015	0.015
Vertical seepage velocity, v_v (ft/day)	0.013	0.006	0.019	0.019	0.019

The vertical seepage velocities range between about 0.006 and 0.019 ft/day. The horizontal seepage velocity is about 0.12 ft/day. The results show the vertical seepage velocity to be significantly less than the horizontal velocities, in fact between about 5 to 15% of the horizontal velocity. Based on this, the implication of an upward vertical hydraulic gradient between the bedrock and the saprolite with respect to the transport of effluent in the environment is considered of minor significance, and thus a site characteristic is not necessary.

References

1. Heath R.C. (1998). “Basic Ground-Water Hydrology,” U.S. Geological Survey Water-Supply Paper 2220.

2. Freeze, R.A. and Cherry, J.A. (1979). “Groundwater.” Prentice-Hall Inc.

Application Revision

None.

DSER Open Item 2.4-10 (DSER page 2-121)

> The applicant stated that the typical hydraulic gradient of ground water flow across the ESP site to Lake Anna and the WHTF is 0.03 m/m. The applicant based this estimate on only one piezometric head contour map constructed using ground water level observations from March 2003. The applicant stated that this hydraulic gradient is typical of the ESP site, despite seasonal and long-term variation in the ground water regime. However, the applicant should provide data to support this statement and to define the range of seasonal and long-term variation in hydraulic gradient from the ESP site into Lake Anna and the WHTF. This is **Open Item 2.4-10.** The staff intends to identify hydraulic gradient from the ESP site to Lake Anna and the WHTF as a site characteristic in any ESP that might be issued for this application.

Response

Groundwater levels in the observations wells installed at the ESP site were monitored quarterly. A total of 4 rounds of quarterly measurements were obtained in the period beginning December 17, 2002 and ending September 29, 2003, and are provided in SSAR Table 2.4-15. Based on the groundwater levels recorded in the wells on March 17, 2003, a piezometric head contour map was prepared and included as SSAR Figure 2.4-16. Evaluation of the contour map indicated a horizontal hydraulic gradient of 0.028 ft/ft from the center of the ESP site footprint to Lake Anna.

A supplementary round of groundwater level measurements was performed on February 1, 2005 (see the response to DSER Open Item 2.4-7). The groundwater level elevations obtained from the last monitoring round, along with the 4 previous monitoring rounds, are provided on the updated version of SSAR Table 2.4-15 (at the end of the response to DSER Open Item 2.4-7).

In order to determine the range in horizontal hydraulic gradients based on seasonal groundwater level fluctuations recorded during the monitoring period, additional piezometric head contour maps have been prepared using the groundwater level elevations recorded during the four other monitoring rounds. The piezometric head contour maps for these rounds are shown on the attached figures, labeled as GWL Contours 12-17-02, GWL Contours 6-17-03, GWL Contours 09-29-03, GWL Contours 02-01-05).

The configuration of piezometric head contours on each of the maps is very similar, reflecting relatively minor fluctuations in groundwater levels recorded between December 2002 and February 2005. During this period, groundwater levels generally

increased by about 0.2 to 8 ft. Two wells, OW-842 and OW-847, show the greatest increase in groundwater levels of 6.7 and 7.9 ft, respectively.

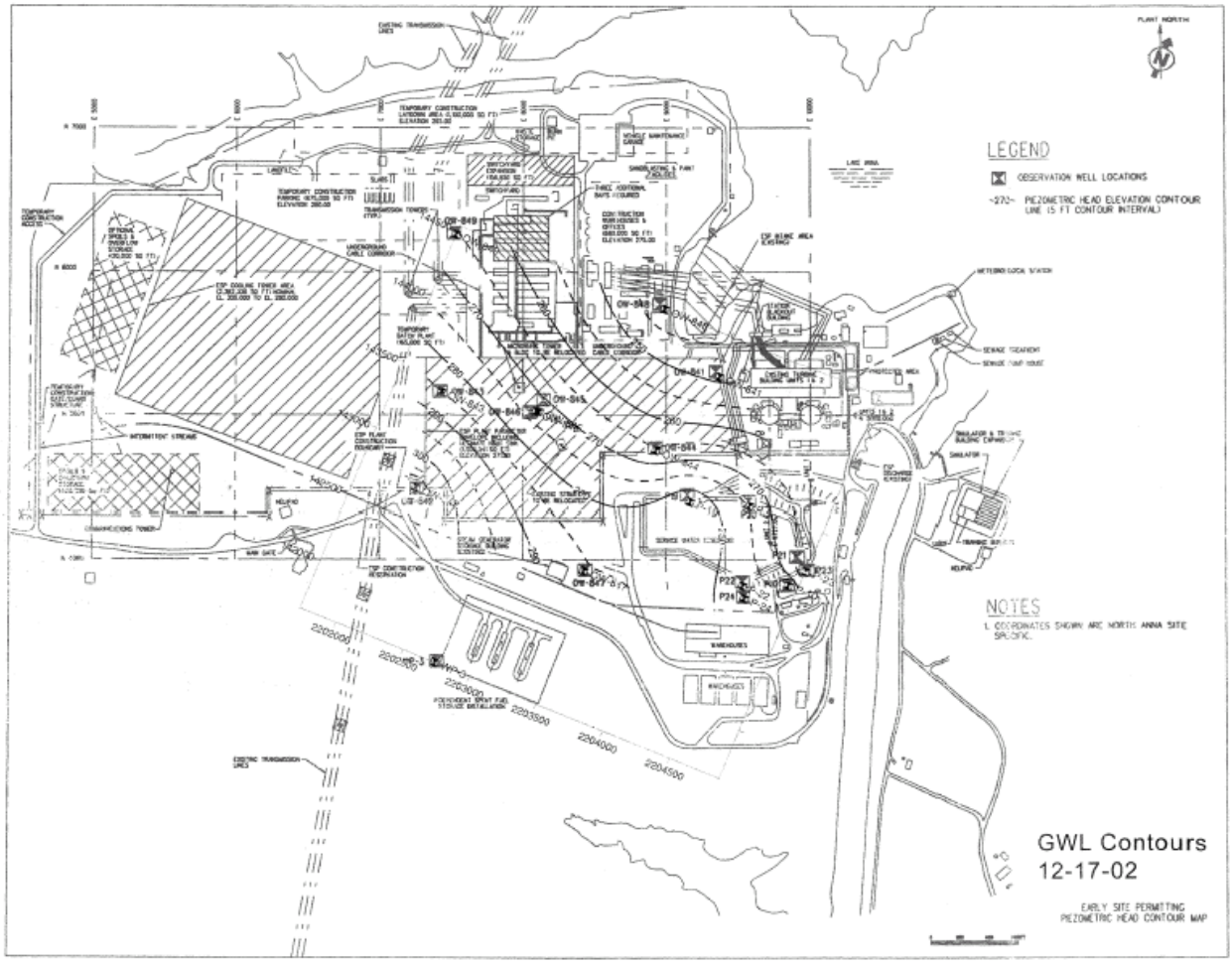
Horizontal hydraulic gradients have been calculated from the center of the ESP site footprint at OW-846 to near the Lake Anna shoreline at OW-848 using the five rounds of groundwater level measurements from December 17, 2002 to February 1, 2005. The resulting horizontal hydraulic gradients range from about 0.027 to 0.029 ft/ft.

Based on the above horizontal hydraulic gradients that reflect seasonal groundwater level fluctuations, the hydraulic gradient of 0.03 ft/ft reported in SSAR Section 2.4.12.1.2 is representative of the hydraulic gradient from the center of the ESP site to Lake Anna. Accordingly, a hydraulic gradient of 0.03 ft/ft will be identified as a site characteristic in SSAR Table 1.9-1.

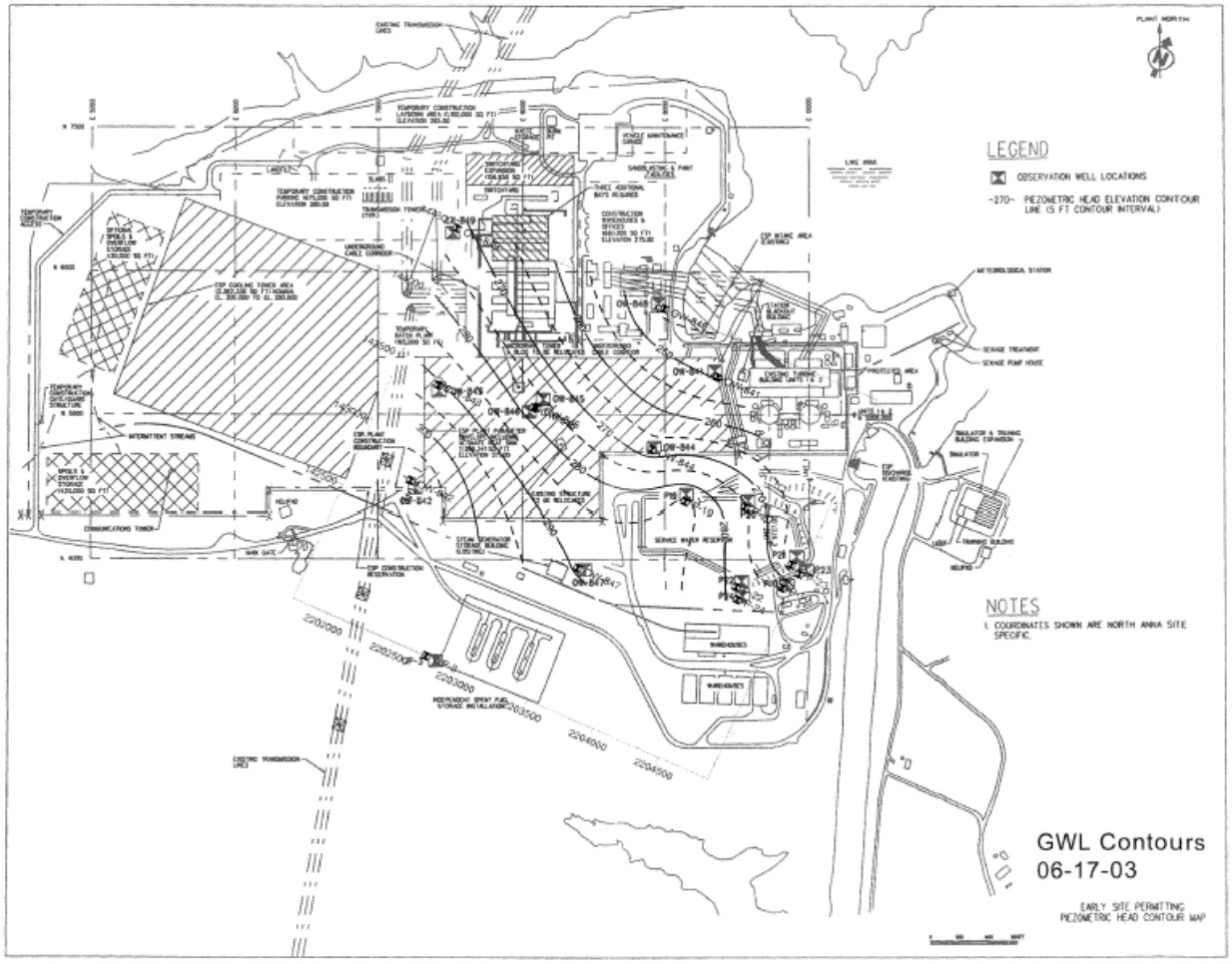
Application Revision

SSAR Table 1.9-1 will be revised to add the following new site characteristic:

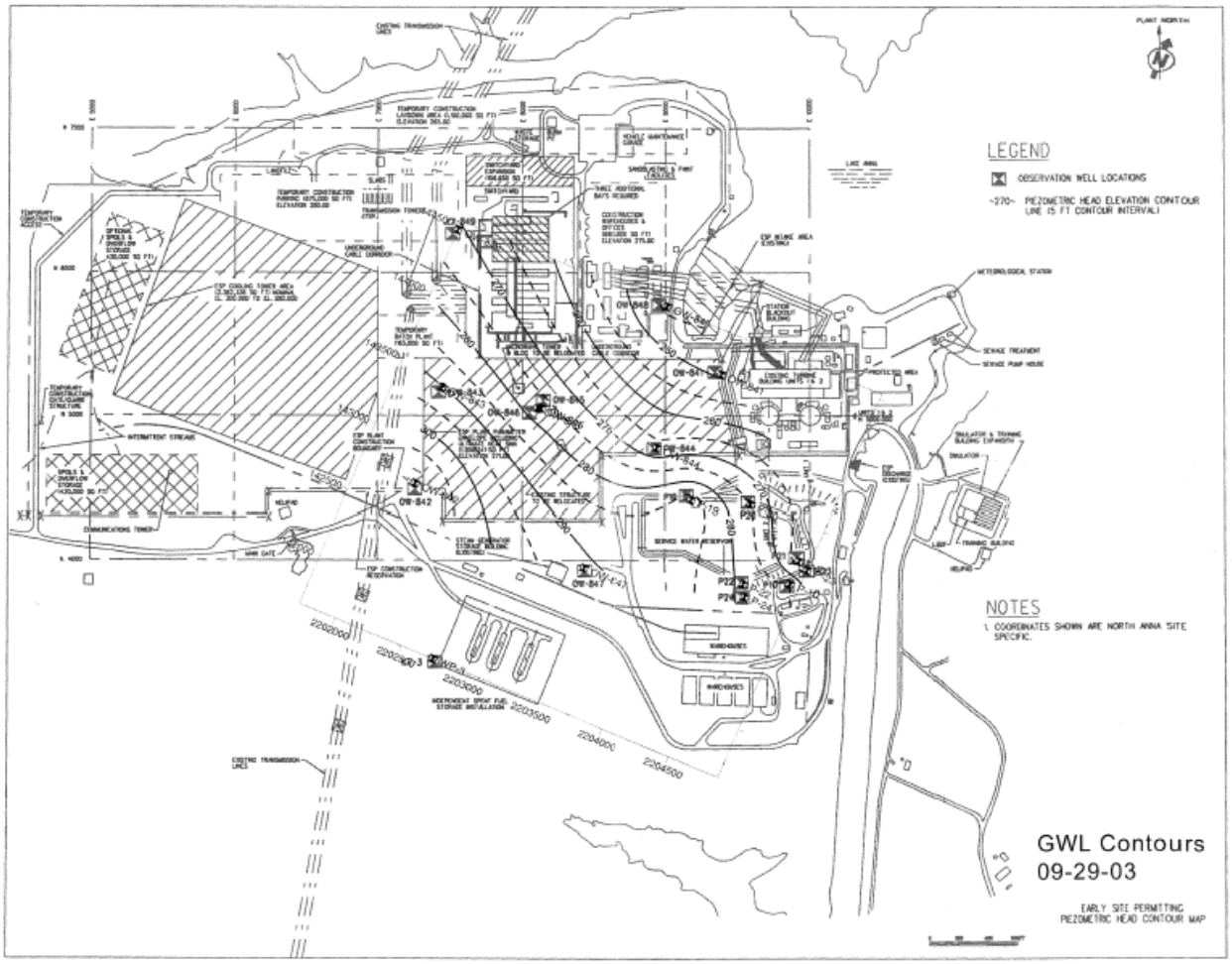
Item	Single Unit/Group Value [Second Unit/Group Value]	Description and References
Hydraulic Gradient	0.03 ft/ft [Same for 2nd unit/group]	▪ Hydraulic gradient used to assess groundwater flow across the ESP site to Lake Anna ▪ Refer to Section 2.4.12.1.2



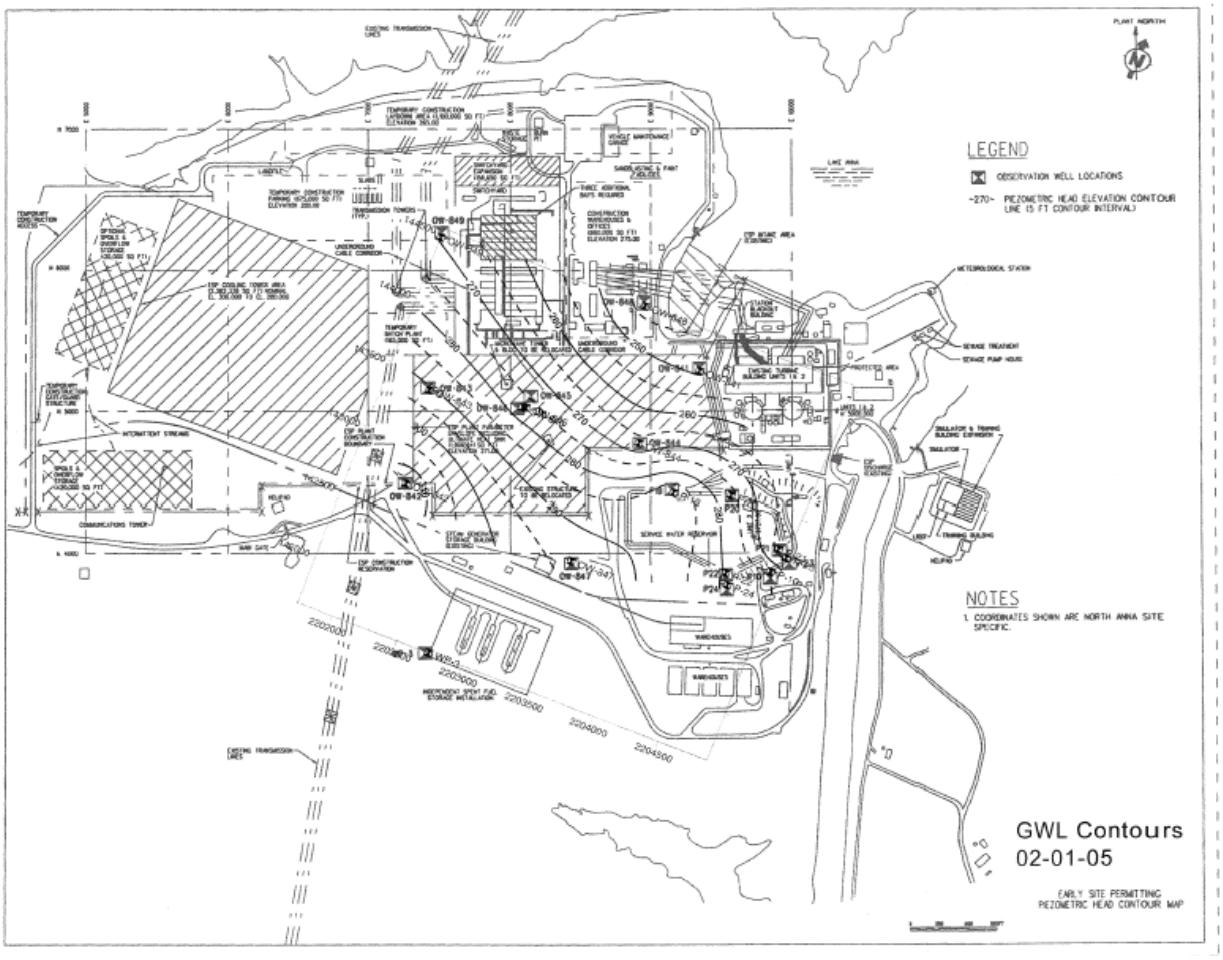
GWL Contours 12-17-02



GWL Contours 06-17-03



GWL Contours 09-29-03



GWL Contours 02-01-05

DSER Open Item 2.4-11 (DSER page 2-121)

> The site suitability evaluation with respect to radionuclide transport characteristic as defined by 10 CFR Part 100.20(c)(3) requires the use of observed site specific parameters important to hydrological radionuclide transport (such as soil, sediment, and rock characteristics, adsorption and retention coefficients, ground water velocity, and distances to the nearest surface body of water) obtained from on-site measurements. The applicant has not provided the onsite measured values of adsorption and retention coefficients for radioactive materials. This is **Open Item 2.4-11.** The staff intends to identify onsite measured values of adsorption and retention coefficients for radioactive materials as a site characteristic.

Response

Site-specific adsorption (distribution) coefficients important to subsurface hydrological radionuclide transport were obtained from on-site measurements of soil characteristics. The process involved characterizing the radionuclide inventory that could potentially be released as a liquid effluent, identifying the radionuclides for which the distribution coefficient (K_d) is a parameter important to subsurface hydrological transport, and determining distribution coefficient values from onsite measurements for the radionuclides for which the distribution coefficient is an important parameter. The various steps of the process are described below.

The radionuclide inventory was estimated from information included in the AP1000 Design Control Document, Tier 2, Table 12.2-9 (Sheet 4) (for the effluent holdup tank liquid phase and the waste holdup tank) (Reference 1) and in the ABWR Standard Safety Analysis Report, Table 12.2-13a (for the low conductivity waste collection tank) (Reference 2), both of which list the radionuclides that are expected to be present in their liquid radwaste systems and their corresponding activities. From these liquid radwaste inventories, a composite list of radionuclides and activities was generated, using the more conservative activity from the two reactor designs.

The radionuclides on the list described above were then screened to identify those for which the distribution coefficient would be a parameter important to subsurface hydrological transport. This was accomplished by assuming an instantaneous release of the radwaste inventory to the saturated zone and then accounting for the radioactive decay that would occur during transport from the point of release to Lake Anna. For screening purposes, the distribution coefficient was assumed to be zero and no credit for adsorption or retardation was taken in estimating the saturated zone travel time. The groundwater travel time from the center of the ESP site to Lake Anna was estimated to be about 16 years based on data included in SSAR Section 2.4.12.1.2. (This travel time

calculation used the maximum observed hydraulic conductivity of 3.4 ft/day, a horizontal hydraulic gradient of 0.03 ft/ft, an effective porosity of 0.33, and a travel distance of 1800 ft.) The activities remaining after 16 years of decay were then compared to the values identified in 10 CFR Part 20, Appendix B, Table 2, Column 2. If the remaining activity for a given radionuclide was less than its 10 CFR Part 20 value, then the distribution coefficient was considered to be not important to subsurface hydrological transport for that particular radionuclide. If the remaining activity was greater that its 10 CFR Part 20 value, the distribution coefficient was considered important to subsurface hydrological transport and the radionuclide was retained for further evaluation. The radionuclides retained in this case include Fe-55, Co-60, Sr-90, Cs-134, and Cs-137.

Distribution coefficients for each of the retained radionuclides were obtained using onsite soil measurements. Distribution coefficients are dependent on soil's physical and chemical characteristics. The physical and chemical properties of the ESP site soils have been characterized as part of the ESP subsurface investigation. SSAR Section 2.5.4.2.2 indicates that about 75% of the Zone IIA saprolites are coarse-grained materials (sands), while about 25% are fined-grained materials (silts and clays). The coarse-grained are composed of 30-40% quartz, 20-30% microline, 25-40% clay minerals, and 5-20% mica. Although the clay content of the fine-grained soils was not determined, it is reasonable to assume 50-80% clay minerals. The site soils therefore have relatively high clay content. For some elements, distribution coefficients are dependent on the pH. SSAR Section 2.5.4.2.5 indicates that the pH of the Zone IIA saprolites ranges from 5.7 to 6.9.

Based on the onsite soil measurements described above, and considering the possibility that fill with differing characteristics could be imported during construction, distribution coefficients were determined for Co, Cs, Fe, and Sr. Distribution coefficients for Co and Fe were obtained from Sheppard and Thibault (Reference 3) by selecting the soil type that yields the most conservative value. In the case of Co, the K_d value for sand of 60 cm^3/g was chosen, while the K_d value for clay of 165 cm^3/g was selected for Fe. The K_d value for Cs was obtained from EPA (Reference 4). To ensure conservatism, a K_d value of 30 cm^3/g was selected, which represents lower end of the range for soils having a clay content in the range of 4-20% and a mica content less than 5%. A K_d value of 15 cm^3/g for Sr was obtained from EPA (Reference 4) by conservatively assuming 4-20% clay content and a pH between 5 and 8 and picking the lower end of the range. These values are summarized in the table below. Distribution coefficients for some other elements, namely Mn, Ru, and Zn, have also been included to allow for the possibility that the saturated zone travel time could be less than 16 years if the release were to occur near the edge of the ESP site footprint as opposed to the center. These values were determined in a manner similar to Co and Fe. The K_d values included in the table below will be identified as site characteristics in SSAR Table 1.9-1.

Radionuclide	Distribution Coefficient K_d (cm^3/g)	Source
Mn-54	50	Reference 3
Fe-55	165	Reference 3
Co-60	60	Reference 3
Zn-65	200	Reference 3
Sr-90	15	Reference 4
Ru-106	55	Reference 3
Cs-134	30	Reference 4
Cs-137	30	Reference 4

References

1. AP-1000 Document No. APP-GW-GL-700, AP1000 Design Control Document, Tier 2 Material, Westinghouse, Revision 2, 2002.

2. Document 23A6100, *ABWR Standard Safety Analysis Report,* General Electric, Revision 8.

3. Sheppard, M. I., and D. H. Thibault, "Default Soil Solid/Liquid Partition Coefficients, K_ds, for Four Major Soil Types: A Compendium," Health Physics, 59, 1990.

4. U. S. Environmental Protection Agency (EPA), "Understanding Variation in Partition Coefficient, K_d, Values, Volume II: Review of Geochemistry and Available K_d Values for Cadmium, Cesium, Chromium, Lead, Plutonium, Radon, Strontium, Thorium, Tritium (^3H), and Uranium," EPA 402-R-99-004B, August 1999.

Application Revision

SSAR Table 1.9-1 will be revised to add the following new site characteristic:

Item	Single Unit/Group Value [Second Unit/Group Value]	Description and References
Distribution Coefficients (K_d)		▪ Distribution coefficients used to assess subsurface hydrological radionuclide transport ▪ Refer to Section 2.4.13
▪ Mn-54	50 cm^3/g [Same for 2nd unit/group]	
▪ Fe-55	165 cm^3/g [Same for 2nd unit/group]	
▪ Co-60	60 cm^3/g [Same for 2nd unit/group]	
▪ Zn-65	200 cm^3/g [Same for 2nd unit/group]	
▪ Sr-90	15 cm^3/g [Same for 2nd unit/group]	
▪ Ru-106	55 cm^3/g [Same for 2nd unit/group]	
▪ Cs-134	30 cm^3/g [Same for 2nd unit/group]	
▪ Cs-137	30 cm^3/g [Same for 2nd unit/group]	

SSAR Section 2.4.13 will be revised to add the following new paragraphs at the end of the section:

Site-specific distribution coefficients (K_d's) important to subsurface hydrological transport have been established for use in any future assessment of accidental releases of liquid effluents to ground and surface waters. Values were obtained from on-site measurements of soil characteristics. The process involved characterizing the radionuclide inventory that could potentially be released as a liquid effluent, identifying the radionuclides for which the distribution coefficient is a parameter important to subsurface hydrological transport, and determining distribution coefficient values from onsite measurements for the radionuclides for which the distribution coefficient is an important parameter. The various steps of the process are described below.

The radionuclide inventory was estimated from information included in the AP1000 Design Control Document (Reference 62), for the effluent holdup tank liquid phase and the waste holdup tank, and in the ABWR Standard Safety Analysis Report (Reference 63), for the low conductivity waste collection tank, both of which list the radionuclides that are expected to be present in their liquid radwaste systems and their corresponding activities. From these liquid radwaste inventories, a composite list of radionuclides and activities was generated, using the more conservative activity from the two reactor designs.

The radionuclides on the list described above were then screened to identify those for which the distribution coefficient would be a parameter important to subsurface hydrological transport. This was accomplished by assuming an instantaneous release of the radwaste inventory to the saturated zone and then accounting for the radioactive decay that would occur during transport from the point of release to Lake Anna. For screening purposes, the distribution coefficient was assumed to be zero and no credit for adsorption or retardation was taken in estimating the saturated zone travel time. The groundwater travel time from the center of the ESP site to Lake Anna was estimated to be about 16 years based on data included in Section 2.4.12.1.2. (This travel time calculation used the maximum observed hydraulic conductivity of 3.4 ft/day, a horizontal hydraulic gradient of 0.03 ft/ft, an effective porosity of 0.33, and a travel distance of 1800 ft.) The activities remaining after 16 years of decay were then compared to the values identified in 10 CFR Part 20, Appendix B, Table 2, Column 2. If the remaining activity for a given radionuclide was less than its 10 CFR Part 20 value, then the distribution coefficient was considered to be not important to subsurface hydrological transport for that particular radionuclide. If the remaining activity was greater that its 10 CFR Part 20 value, the distribution coefficient was considered important to subsurface hydrological transport and the radionuclide was retained for further evaluation. The radionuclides retained in this case include Fe-55, Co-60, Sr-90, Cs-134, and Cs-137.

Distribution coefficients for each of the retained radionuclides were obtained using onsite soil measurements. Distribution coefficients are dependent on soil's physical and chemical characteristics. The physical and chemical properties of the ESP site soils have been characterized as part of the ESP subsurface investigation. Section 2.5.4.2.2 indicates that about 75% of the Zone IIA saprolites are coarse-grained materials (sands), while about 25% are fined-grained materials (silts and clays). The coarse-grained are composed of 30-40% quartz, 20-30% microline, 25-40% clay minerals, and 5-20% mica. Although the clay content of the fine-grained soils was not determined, it is reasonable to assume 50-80% clay minerals. The site soils therefore have relatively high clay content. For some elements, distribution coefficients are dependent on the pH. Section 2.5.4.2.5 indicates that the pH of the Zone IIA saprolites ranges from 5.7 to 6.9.

Based on the onsite soil measurements described above, and considering the possibility that fill with differing characteristics could be imported during construction, distribution coefficients were determined for Co, Cs, Fe, and Sr. Distribution coefficients for Co and Fe were obtained from Sheppard and Thibault (Reference 64) by selecting the soil type that yields the most conservative value. In the case of Co, the K_d value for sand of 60 cm^3/g was chosen, while the K_d value for clay of 165 cm^3/g was selected for Fe. The K_d value for Cs was obtained from EPA (Reference 65). To ensure conservatism, a K_d value of 30 cm^3/g was selected, which represents lower end of the range for soils having a clay content in the range of 4-20% and a mica content less than 5%. A K_d value of 15 cm^3/g for Sr was obtained from EPA (Reference 65) by conservatively assuming 4-20% clay content and a pH between 5 and 8 and picking the lower end of the range. These values are summarized in Table 2.4-20. Distribution coefficients for some other elements, namely Mn, Ru, and Zn, have also been included to allow for the possibility that the saturated zone travel time could be less than 16 years if the release were to occur near the edge of the ESP site footprint as opposed to the center. These values were determined in a manner similar to Co and Fe.

SSAR Section 2.4 References will be revised to add the following new references:

62. AP-1000 Document No. APP-GW-GL-700, AP1000 Design Control Document, Tier 2 Material, Westinghouse, Revision 2, 2002.

63. Document 23A6100, ABWR Standard Safety Analysis Report, General Electric, Revision 8.

64. Sheppard, M. I., and D. H. Thibault, Default Soil Solid/Liquid Partition Coefficients, K_ds, for Four Major Soil Types: A Compendium, Health Physics, 59, 1990.

65. U. S. Environmental Protection Agency (EPA), Understanding Variation in Partition Coefficient, K_d, Values, Volume II: Review of Geochemistry and Available K_d Values for Cadmium, Cesium, Chromium, Lead, Plutonium, Radon, Strontium, Thorium, Tritium (^3H), and Uranium, EPA 402-R-99-004B, August 1999.

The following new table will be added to SSAR Section 2.4:

Table 2.4-20 Distribution Coefficients Important to Subsurface Hydrological Transport

Radionuclide	Distribution Coefficient K_d (cm^3/g)	Source
Mn-54	50	Reference 64
Fe-55	165	Reference 64
Co-60	60	Reference 64
Zn-65	200	Reference 64
Sr-90	15	Reference 65
Ru-106	55	Reference 64
Cs-134	30	Reference 65
Cs-137	30	Reference 65

DSER Open Item 2.5-2 (DSER page 2-167)

The staff focused its review of SSAR Section 2.5.2.5, "Seismic Wave Transmission Characteristics of the Site," on the applicant's incorporation of the seismic wave transmission characteristics of the material overlying the base rock at the site into the determination of the SSE. SSAR Section 2.5.4.7 provides a description of the transmission characteristics of the site material. According to the applicant's responses to RAIs 2.5.2-1(c) and 2.5.2-8, the applicant's SSE represents the ground motion at a depth well below the ground surface. However, 10 CFR 100.23(d)(1) states the following:

The Safe Shutdown Earthquake Ground Motion for the site is characterized by both horizontal and vertical free-field ground motion response spectra at the free ground surface.

As explained in more detail below, the staff has determined that the applicant's SSE does not represent the free-field ground motion at the free ground surface.

Figure 2.5.2-5, which reproduces SSAR Figure 2.5-62, shows that the shear wave velocity values for the ESP site reach a value of about 2500 feet per second (ft/s) at a depth of 60 feet.

This shear wave velocity value is well below that of the hard rock conditions (Vs = 9200 ft/sec) assumed by the EPRI 2003 study for CEUS ground motion models. In addition, the applicant did not make shear wave velocity measurements at a depth greater than 65 feet. Thus, the hard rock shear wave velocity value of 9200 ft/s may not be reached at the ESP site until a considerable depth below the ground surface. According to SSAR Figure 2.5-62, from the ground surface to a depth of 30 feet, the shear wave velocity at the ESP site varies from 600 ft/s to about 1300 ft/s. The applicant needs to incorporate these lower shear wave velocities, as well as other subsurface material properties and their uncertainties, into the determination of the ESP site SSE. In addition, the applicant should provide the site amplification or transfer function for the staff to review. The staff needs this information to determine that the applicant has provided an SSE that meets the requirements of Appendix S to 10 CFR Part 50 and 10 CFR 100.23, which define the SSE as "free-field ground motion response spectra at the free ground surface." This is **Open Item 2.5-2.**

<u>Response</u>

A description of the planned approach to respond to this open item was provided in our February 18, 2005 letter (Reference 1). The information below provides a description of

the input parameters and actual analyses that are being performed. We anticipate completing the analysis and submitting the final results and SSAR changes by March 31, 2005.

In response to this DSER Open Item, we are performing analyses to estimate the SSE ground motion at a control point at the top of a hypothetical outcrop of Zone III-IV material. The shear wave velocities for the Zone III-IV material range from 2500 to 4500 ft/sec, with a best estimate wave velocity of 3300 ft/sec. See SSAR Table 2.5-45. 3300 ft/sec will be used in the control point SSE analysis. The elevation of the top surface of the Zone III-IV material varies across the site (see SSAR Figures 2.5-57 and 2.5-58). The top of the Zone III-IV material will be chosen to be at a representative elevation of 250 ft.

The SSE motion at three other control points will also be estimated:

- It is likely that any new reactor building (containment) structure will be founded at an elevation lower than Elevation 250 ft, with the present best estimate of foundation elevation at about 205 ft. Therefore, the SSE ground motion will be estimated at a hypothetical rock outcrop at an elevation of 205 ft. A shear wave velocity of 5200 ft/sec will be used at El. 205 ft. (It is noted that the shear wave velocity of 5200 ft/sec is different than that specified in Reference 1 at this elevation. The 5200 ft/sec shear wave velocity is a result of further detailed evaluations of boring data.) The purpose of this control point is to provide an SSE ground motion that is closer (both in shear wave velocity and in elevation) to the ground motion that would eventually be used in detailed structural design.

- SSAR Table 2.5-45 shows the shear wave velocities for the Zone IV material range from 4000 to 8000 ft/sec, with a best estimate shear wave velocity of 6300 ft/sec. The estimated representative elevation for a shear wave velocity of 6300 ft/sec is 180 ft. The SSE ground motion will be estimated at a hypothetical rock outcrop of Zone IV material at Elevation 180 ft.

- Finally, the SSE ground motions currently shown in the SSAR will be identified for hard rock material with a shear wave velocity of approximately 9200 ft/sec.

The following is a description of the analysis steps to calculate ground motions at the given elevations.

- The site-specific rock properties and uncertainties in these properties used in the analysis will be documented. These properties are defined for rocks under the site with shear wave velocities increasing from 9200 ft/sec at depth to 3300 ft/sec at a representative elevation of 250 ft. The rock properties will be selected using

available site-specific information as well as extrapolated and/or supplemental generic material property estimates where site-specific data are lacking.

- A suite of 50 alternative rock columns will be generated using the above characterization of site-specific rock properties and uncertainties.

- Two seed time histories will be selected for earthquakes with magnitudes and distances that are appropriate for both the 1 to 2.5 Hz and 5 to 10 Hz (low-frequency and high-frequency) "controlling earthquakes" of RG 1.165. The magnitudes and distances of these earthquakes are specified in the SSAR. The specific seed time histories that will be used are the two used in the current SSAR evaluation of the response of soil and rock to dynamic loading (SSAR Section 2.5.4.7) plus two more taken from the appropriate distance-magnitude bins of NUREG/CR-6728.

- From these seed time histories, spectrum-compatible time histories will be generated for both the high frequency and low frequency scaled spectra used to develop the current SSAR SSE following the spectral matching procedure given in NUREG/CR-6728 (a total of 4 spectral matches).

- SHAKE analyses will be performed for each combination of the four spectrum-compatible time histories and the 50 randomized rock columns. The Zone III-IV hypothetical rock outcrop control point SSE ground motion, at elevation 250 ft, will be defined as the mean of these 200 analysis results (4 time histories times 50 randomizations). The frequency-dependent ratios of the spectral accelerations of the current SSAR SSE and this Zone III-IV hypothetical rock outcrop control point SSE will be the transfer function of the site-specific materials above the "hard rock" SSE.

- During these SHAKE runs, hypothetical rock outcrop motions will also be compiled for the control points at Elevation 205 ft and Elevation 180 ft. A mean response spectrum will also be defined at these points.

- It is anticipated that strains within the randomized rock columns will be low and well within the range for which material properties are independent of strain level. Inter-column strains that are calculated as part of the SHAKE runs will be examined and specified in support of this assumption.

- The implications of the Zone III-IV hypothetical rock outcrop control point SSE ground motions on the liquefaction potential and slope stability analyses will be re-evaluated and these analyses revised if appropriate.

- The amplification factors (transfer functions) between the 9200 ft/sec, 6300 ft/sec, 5200 ft/sec, and 3300 ft/sec hypothetical outcrop horizons will be specified, which are the frequency-dependent ratios of response spectral amplitudes between these horizons. These ratios depend only on the material properties of the site profile between the 9200 ft/s horizon and overlying hypothetical rock outcrop.

Application Revisions

Revisions to the SSAR will be provided in our March 31, 2005 final response.

DSER Open Item 13.3-3 (DSER pages 13-33 and 13-34)

In SSAR Sections 13.3.2.2.2.h.1 through 13.3.2.2.2.h.2, the applicant offered a slightly revised statement of the general guidance criteria from NUREG-0696 for the TSC, OSC and EOF, when compared to that provided above. In order for the NRC staff to determine whether major feature H is acceptable, the applicant needs to address the adequacy of the facilities and related equipment in support of emergency response, and to address, with specificity, such facility and equipment features as location, size, structure, function, habitability, communications, staffing and training, radiological monitoring, instrumentation, data system equipment, power supplies, technical data and data systems, and record availability and management. This is **Open Item 13.3-3.**

Response

Dominion is unable to provide the level of detail requested by the NRC staff to support its review of Major Feature H related to the Technical Support Center, Operational Support Center and Emergency Operations Facility. Therefore, Dominion withdraws its request that this major feature be evaluated as part of the North Anna ESP application.

Application Revision

SSAR Section 13.3.2.2.2.h will be revised to indicate that Dominion does not seek approval of this major feature.

DSER Open Item 13.3-6 (DSER page 13-42)

> The information regarding the ETE requested in RAI 13.3-15, to which the applicant has yet to respond, is **Open Item 13.3-6**. The NRC and FEMA will review the responses to these RAIs and will describe the results of that review in the FSER. Section 13.3.2 of this SER discusses the associated description of contacts and arrangements made with offsite agencies with emergency planning responsibility.

Response

RAI 13.3-15 consists of subparts, labeled “a” through “s.” Each subpart question and a response is provided on the following pages.

RAI 13.3-15a

ETE Table 8 (Roadway Characteristics) identifies the road segments and characteristics within the plume exposure EPZ. Please provide the associated assumptions and data on road capacities and travel times.

Response to 13.3-15a

The ETE analysis used the Evacuation Simulation Model (ESIM), the core component of the Oak Ridge Evacuation Modeling System (OREMS), to establish the ETEs. The Center for Transportation Analysis developed OREMS at the Oak Ridge National Laboratory (ORNL) for the Federal Emergency Management Agency (FEMA) and the US Army in support of the Chemical Stockpile Emergency Preparedness Program (CSEPP). Based on separate correspondence between the U.S. Department of Transportation (DOT) Chief Traffic Systems Branch and the Project Director Chemical Stockpile Emergency Preparedness Program (CSEPP) Technical Training; RE: The Oak Ridge Evacuation Modeling System (OREMS), dated 4/20/1995, the U.S. DOT "endorses the continued development and usage of OREMS as an emergency evacuation planning modeling system"

ESIM is a node based macro-simulation model. The network data is defined in terms of "nodes" and not explicit "links." According to the OREMS 2.50 User's Guide, the input data required by ESIM includes the following:

- Upstream node identifier & Downstream node identifier; defining a link in the network
- Length of the link
- Number of through and turn pocket lanes
- Protected and prohibited turn movements
- Lane and link connections
- Start-up lost time experienced by the first vehicle in the queue
- Average delay for a vehicle in a queue (queue discharge headway)
- Link speed limit
- Road type (freeway, urban)

The officially designated evacuation routes for the NAPS Protective Action Zones were modeled as depicted in the 2001 Nuclear Emergency Information Calendar. A complete review of the evacuation roadway network was performed by driving the roads and collecting information on speed limits, lane numbers, signalization (location of signals, STOP & YIELD signs), and road geometry (lane divisions, turn movements etc). Also, the data was verified against digital spatial data representing roads available from the US Census Bureau. This road data is included in the Census Bureau's Topologically Integrated Geographic Encoding and Reference (TIGER) files.

Capacities on the evacuation network were calculated internally within ESIM based on the link data described above. The OREMS 2.50 User's Guide states; "a comprehensive capacity estimation model has been included in ESIM models. This model produces, through iterations, accurate estimates of service rates (capacities) for each link of the highway network by taking into account the assigned volumes and type of traffic control at each intersection. The solution procedure used in the capacity model is rapid, accurate, and unconditionally convergent."

Appendix A of the OREMS 2.50 User's Guide states: "The capacity of a link (input as the mean discharge headway) is estimated based on the procedures described in the Highway Capacity Manual (TRB, 1985). The impact of restricted highway geometry is taken into account in computing the capacity of highway sections." Use of the Transportation Research Board's (TRB's) Highway Capacity Manual (HCM) is a standard practice in transportation planning and modeling. It is widely used for estimating the capacity and determining the level of service for different types of roadways (freeway, urban, rural) and intersections (pre-timed, actuated, STOP and YIELD signs). The capacity of a road segment or intersection depends on prevailing roadway, traffic, and control conditions. Some of the roadway conditions that affect capacity of a road segment/intersection are number of lanes, design speed, lane width, shoulder width, median type, terrain, and grade.

The OREMS 2.50 User's Guide also states; "the assignment submodel uses an impedance function to capture the increase in travel time with increasing congestion. The assignment submodel in OREMS can use either the Bureau of Public Records (BPR) function or the modified Davidson's queuing function as the impedance function." This study used the Bureau of Public Records function. The BPR function calculates the travel time on a link based on free flow travel time on the link, volume of flow on the link, capacity of the link, and link saturation flow rate.

RAI 13.3-15b

> ETE Table 9 (Summary of Results of Evacuation Time Analysis) provides an overall summary of evacuation times. Please provide any traffic control measures necessary to direct the public out of the EPZ. Also, please discuss whether the ETE depends on these measures being in place.

Response to 13.3-15b

A critical part of the evacuation planning process is the development of a traffic control plan and a traffic management plan. Besides traffic signs and signals, other traffic control measures include the locations and roles of traffic control points (TCP) and

access control points (ACP) during an evacuation. These latter measures were not explicitly modeled, since ETE studies are generally used to identify potential traffic congestion and to develop plans for traffic management and the use of traffic control personnel during an evacuation.

In the NAPS area there is minimal signalization. Where traffic signals on major evacuation routes existed, they were modeled as green. The roads crossing at these signals were modeled as having “YIELD” or “STOP” signs. Roadway sections at which the main evacuation routes had STOP or YIELD signs were modeled accordingly. This assumption is prudent in an evacuation scenario, since regular traffic control strategies may not be applicable during an evacuation.

The ETEs do not explicitly depend on the implementation of the emergency response traffic control outlined in the counties’ RERPs. It is expected that ETEs will not increase due to their implementation. Some aspects of the RERP traffic control measures (TCPs and ACPs) are implicit in the assumptions of the ETE modeling.

TCPs can facilitate in resolving the congestion related delays at intersections, thereby cutting down the travel times of evacuees along specific evacuation routes. Generally, TCPs need to be in place prior to evacuation to effectively manage traffic flow. The ETE analysis assumes that travel on the major evacuation routes is not hindered due to signalization. This is consistent with TCPs providing traffic direction applicable to maintaining smooth traffic flow. However, it is possible for traffic to maintain smooth flow without these TCPs, especially in areas with minimal signalization, such as the NAPS area.

The ETE analysis assumes that there is no ingress of additional vehicles into the EPZ. This is consistent with applicably placed ACPs.

RAI 13.3-15c

> The County radiological emergency response plans (RERPs) identify numerous locations for traffic control. Please discuss the resources and time necessary to implement these measures, if needed, in support of evacuation.

<u>Response to 13.3-15c</u>

The ETE study required no explicit assumptions regarding the timing or implementation of traffic control measures by local emergency response organizations. The ETEs provided in the Evacuation Time Estimates for the North Anna Power Station and Surrounding Jurisdictions (2001) report were developed to be used as a basis for the development of emergency response plans including traffic control. It is expected that

the implementation of emergency traffic control would have a beneficial impact on the ETEs provided in this report.

Some implicit assumptions pertaining to traffic flow are consistent with TCPs and ACPs being in place. These include the assumption that the few traffic signals on major evacuation routes would not hinder traffic on those routes, and that no additional vehicles would enter the NAPS areas being evacuated.

Appendix 4 of Louisa County Radiological Response Plan (Sheriff Department Procedure) provides detailed discussion on the standard operating procedure (SOP) of the Sheriff's Department in the event of an evacuation due to a radiological event.

Evacuation time estimates are used by local emergency management agencies to develop traffic control and traffic management plans to facilitate the evacuation process.

Louisa County Radiological Response Plan indicates law enforcement agencies have the leading role in the planning and implementation of traffic and access control measures along the route, while the state department of transportation provides barricades for traffic and access control operations at pre-established locations as outlined in the County SOPs, or at additional locations as identified during the emergency. The plans do not contain any explicit discussion of the time frame assumed for implementing the traffic and access control measures. The only mention of approximate time of activation of traffic control measures in Appendix 4 of the Louisa County Radiological Response Plan reads:

> If an evacuation has been recommended, instruct the Highway Department personnel to deliver barriers to specific locations and instruct traffic control personnel to proceed to their assigned traffic control points after acquiring their dosimeter. *Wait for an evacuation order to activate the traffic control points, unless instructed otherwise by the Emergency Management Coordinator*.

Sections III-B and IV-B of the Louisa County Radiological Response Plan also reference the FEMA guidance regarding completion of route alerting within 45 minutes of the Governor's order to alert the public. In order to ensure that the traffic and access control measures are effective, it can be assumed that the TCPs and ACPs should be ready for operations before the evacuation order is issued to the public. There is no explicit mention of the resource requirements for implementation of traffic and access control measures in any of the plans. The number of locations where TCPs and ACPs need to be manned and the time of the incident onset determine the resource needs.

RAI 13.3-15d

ETE Section 3.1.2 (Key Evacuation Parameters) provides information on the assumptions of adverse weather conditions, including using snow and ice as the adverse weather conditions, and a reduced road capacity of 40%. Please provide additional information on the assumptions used, including consideration of (1) any additional time that may be needed for evacuation preparation (such as putting on tire chains), (2) any reduction in both road capacity and travel times, and (3) resources and time that may be necessary for clearing the driveways and major roadways of snow and ice to support the evacuation.

Response to 13.3-15d

The ETE study was intended to consider general evacuation scenarios due to a radiological event at NAPS. The distribution of mobilization times that was used to generate the loading time distribution, simulating evacuees getting onto the network, is based on real-life evacuation observations (reference: ORNL-6615, Evaluating Protective Actions for Chemical Agent Emergencies, 1990, G. O. Rogers, primary author). The underlying assumptions for this distribution include the general activities that evacuees would do in preparing for an evacuation. However, there may be instances where some specific activities/tasks, such as putting on tire chains or clearing off driveways, would be performed by a small number of evacuees before they start evacuating. It would not be prudent to generalize their behavior across the entire group of evacuees. It is assumed that driveways and roads are cleared on a regular basis and snow chains are applied seasonally; or immediately following an isolated incident of bad weather. The ETEs are intended to be based on general bad weather conditions. The Rogers' loading curve is consistent with general behavior of the evacuating population.

All capacity related calculations are done within the simulation software as described in the OREMS 2.50 User's guide. Bad weather in the area around NAPS was assumed to be predominantly due to snow and ice. To accommodate the impact of bad weather on the ETEs, the speed limits were reduced by 40 %. This effectively reduced the road capacities by about 25 %. This is consistent with research that concludes that in a snow event, drivers may reduce their velocity by nearly 40 percent, which can result in a 25 to 30 percent reduction in capacity (reference: Where the Weather Meets the Road: A Research Agenda for Improving Road Weather Services; Transportation Research Board (TRB), Board on Atmospheric Services, 2004). Weather-related capacity reductions of 20–25% are generally used in current evacuation studies under bad weather roadway conditions (reference: NUREG/CR-4831; PNL-7776, State of the Art in Evacuation Time Estimate Studies for Nuclear Power Plants, 1992. The final Evacuation Time Estimates for the North Anna Power Station and Surrounding

Jurisdictions (2001) report, however, misquoted the term “speed” as “capacity” in discussing the reduction of roadway capacities due to bad weather.

RAI 13.3-15e

> ETE Section 3.1.1 (Loading of the Evacuation Network) identifies that evacuation network loading is derived from data presented in the 1990 ORNL study, "Evaluating Protective Actions for Chemical Agent Emergencies." ETE Section 3.1.2 provides distribution curves that are derived from the study, which reflect chemical releases that have an immediate threat to life. Please describe how the ETE uses this information to address the development of trip generation times for a radiological release that such evacuees would have sufficient time to mobilize.

Response to 13.3-15e

The underlying assumption regarding the applicability of the Rogers’ mobilization curves in the ETE study is that public perception of radiological emergencies differs from the actual characteristic of such an event. The familiarity of the hazard and the social assessment of the risks associated with the hazards are among the underlying forces that guide the decision making process in an evacuation scenario. People are more likely to respond to calls for evacuation when the assessment of threat in the community is high and dangers to life and property are recognized. The reality may be different. The alarm associated with social response in a radiological emergency makes the use of Rogers’ mobilization curves prudent for the ETE study.

It should also be noted that these curves were developed from the data collected from real-life evacuations in response to actual events. No similar study developed specifically for radiological events is readily available. Therefore, the widely accepted Rogers’ mobilization curves were used for this study. The implications of assuming public behavior in absence of real data are unknown. Actual data taken from a somewhat similar real-life scenario and similar public response are more applicable than assumptions about how people might behave.

RAI 13.3-15f

ETE Section 3.1.2 states that the assumption of "user equilibrium" is applied to account for local residents' knowledge and use of alternate paths to get to the same destination, as specified in the recommended evacuation routes, and that the evacuating population can and will adjust their routes in response to perceived (evacuation) impedance. County RERPs designate traffic control points that may limit user equilibrium. Please clarify how this user equilibrium assumption was modeled, and why it is needed.

Response to 13.3-15f

In transportation planning, the process of route or path selection is referred to as traffic assignment. In most traffic simulation problems, two methodologies can be used for traffic assignment: user equilibrium / user-optimal (UO) or system equilibrium / system-optimal (SO).

The UO methodology minimizes the travel times of individual vehicles (reference: OREMS 2.50 User's Guide). For each origin-destination (O-D) pair, drivers experience similar travel times, and no unused route has a lower travel time (reference: User-Optimal and System-Optimal Route Choices for a Large Road Network, authored by David Boyce, Northwestern University, and Qian Xiong, TJKM Transportation Consultants). The idea behind UO route-choice is that drivers have information concerning their shortest routes from origins to destinations from their day-to-day driving behavior experience.

The SO assignment minimizes the sum of travel times over all vehicles. In the SO case, the route travel times experienced by drivers for a given OD pair are different. Boyce and Xiong performed a route choice analysis using UO and SO options for the network of the Chicago region. The analysis indicates that route choices for individual OD have "losers as well winners in SO case. Some travelers are worse off in the system-optimal solution, in some cases substantially so." SO flows are also harder to impose in practice, since it would be necessary to force some individual drivers to paths with longer travel times than those otherwise selected by the drivers.

Because of these issues, analyses frequently use a UO traffic assignment approach. The traffic simulation software (ESIM) used for the ETE analysis has the option to run an UO or a SO traffic assignment. Due to its practicality and feasibility in real scenarios, the UO traffic assignment was chosen. ESIM simulates the UO traffic assignment based on its in-built algorithm (Frank-Wolfe Algorithm).

It is true that traffic controls in a network can limit user equilibrium if those traffic controls force some vehicles to take routes with longer travel times. The purpose of the counties' RERPs' traffic control points is not to force vehicles to take a particular route with longer travel time, but to maintain reasonable traffic flow. Drivers are generally free to choose their own route based on available routes; therefore use of UO approach is more appropriate.

RAI 13.3-15g

> ETE Section 3.1.2 states that, because the non-vehicle owning population is a small fraction of the total population, and these individuals typically have neighbors with cars, there is no need for special treatment of them in an evacuation analysis. Please describe how the use of neighbors to provide transport to non-auto owning populations affects the traffic loading.

Response to 13.3-15g

The distribution of warning diffusion times was combined with the distribution of mobilization times (generated from the data collected in three meta-study mentioned above) to form a composite traffic loading curve for the general population, including non-auto owning population. Because non-auto owning populations are included in the actual data that is the basis for the loading curves, the traffic loading of non-auto owning population was implicitly represented in this composite traffic loading curve.

Public response to an emergency (chemical, radiological, or explosive) depends on the warning systems in place and the mobilization of people after receiving the warning. As mentioned in the section 3.1.1 of the Evacuation Time Estimates for the North Anna Power Station and Surrounding Jurisdictions (2001) report, "the timing of network loading is derived from data presented in the 1990 ORNL study, "Evaluating Protective Actions for Chemical Agent Emergencies." This data was collected during evacuations executed in response to large-scale chemical spills, and explicitly incorporates the time required for communication of the warning (warning diffusion) and the time required for an individual to respond to the warning (mobilization). The data collected in this meta-study was based on transient, permanent, and special populations and is therefore appropriate to use as 'general' warning diffusion and public mobilization curves for all three population types.

Timing public response to emergency warning has been studied in conjunction with three train derailments, one in Mississauga, Ontario, another Pittsburgh, Pennsylvania, and a third in Confluence, Pennsylvania. In all three emergencies, the principal response for individuals was to evacuate the affected area. In Mississauga, approximately 250,000 people were evacuated over a 4 days period (Burton et al.

1981). In Pittsburgh, about 22,000 people were evacuated (Federal Railroad Administration 1987). In Confluence, all 986 residents were evacuated (PEMA 1987).

RAI 13.3-15h

> ETE Section 4.3 (Estimates for Non-Auto-Owning Population) states that the non-auto owning population is approximately seven to eight percent of the population in Louisa and Spotsylvania Counties, and that it is reasonable to expect that the majority of the population needing transportation will be able to evacuate with neighbors or relatives. Further, the ETE states that any remaining individuals stranded without transportation will be accounted for during the confirmation of evacuation and route alerting via signs to be placed in residents' windows, and that these signs are distributed in public outreach calendars. Please describe the bases for these assumptions, including assurances that this evacuation and confirmation will occur. In addition, for those who may be stranded, please clarify how their accounting is consistent with County RERPs, which identify bus routing for pickup of non-auto owning populations.

Response to 13.3-15h

As stated in Section 2.1 of the Evacuation Time Estimates for the North Anna Power Station and Surrounding Jurisdictions (2001) report, IEM obtained permanent resident population data from Year 2000 census. Emergency management plans produced by counties and the state include results from recent studies of auto ownership for the population surrounding NAPS. IEM used a figure for per capita auto ownership derived from these plans and applied it to the current permanent resident population. The population data for five different counties around NAPS is presented in the Table 3 of the Evacuation Time Estimates for the North Anna Power Station and Surrounding Jurisdictions (2001) report.

The Nuclear Emergency Information Calendars distributed in each of these counties informs the public to assist friends and neighbors without transportation. In addition, the Louisa County Radiological Emergency Response Plan – Basic Plan states that persons without their own means of evacuation will be transported by local government vehicles, non-ambulatory persons will be evacuated by members of the immediate family or friends (if possible), and those requiring transportation will be identified (before or at the time of the emergency) and transportation will be provided. From the county RERPs and public outreach information described above (that is, the Nuclear Emergency Information Calendar), it is assumed that the majority of the population needing transportation will be able to evacuate with neighbors or relatives.

Although county RERPs identify bus routing for pickup of non-auto-owning populations, they also encourage any non-ambulatory persons to evacuate with members of the immediate family or friends, if possible. Similarly, the Nuclear Emergency Information Calendars encourage people without transportation to make plans to ride with a neighbor. For those requiring special assistance, a card is provided in the Nuclear Emergency Information Calendar to alert emergency workers of their need. Therefore, the assumptions taken for non-auto owning population are consistent with the county RERPs and public outreach efforts.

The analysis validated that there were plans in place that would result in confirmation of evacuation. The analysis did not consider deviations from these plans. The execution of existing plans, although not assured, is a reasonable assumption.

RAI 13.3-15i

> Please describe how the analysis of the site-specific permanent population group was modeled in the Evacuation Simulation Model (ESIM), and provide an estimate of the time to evacuate the permanent population group, including car owners and non-car owners. In addition, please describe how projected demography has been taken into consideration in the ETE.

Response to 13.3-15i

As stated in Section 2.1.1 of the Evacuation Time Estimates for the North Anna Power Station and Surrounding Jurisdictions (2001) report, IEM obtained permanent resident population data from the Year 2000 Census. Demographic projections for night-time populations were not necessary. It was assumed that people are located according to the Census at night. Day-time populations were obtained by adjusting night-time populations using data from the U.S. Census Bureau, business location data from the Claritas Corporation, school data from the State of Virginia, and contact with individual facilities.

The night-time populations were higher than the day-time because people tend to work in areas away from the rural NAPS area. Therefore, the night-time population counts were used to as the worst case scenario. The population was then used to calculate the number of vehicles by assigning a loading factor of 2.5 people per vehicle. These vehicles were loaded onto the evacuation network in ESIM from their geographic locations as identified by the Census.

The time to evacuate the general population is presented in the Table 9 of the Evacuation Time Estimates for the North Anna Power Station and Surrounding Jurisdictions (2001) report. The general population includes permanent (both car

owners and non-car owners), transient, and special facility population. The permanent population group was not analyzed separately, since an emergency evacuation involves all segments of the population. The times for the permanent population to evacuate are consistent with the overall ETEs reported.

Because actual Census counts of night-time population were used for the basis of the scenarios, and there are small contributions due to transient populations, demographic projections do not affect the ETEs.

RAI 13.3-15j

> ETE Section 4.4 (Estimates for Special Facilities) states that the schools within ten miles of North Anna Power Station (NAPS) have evacuation resources immediately available. In addition, school evacuations had been included with the general population during the evacuation analysis, and they share the time estimates for the general population. Please provide information on trip generation times from the schools to evacuation locations. Please address whether return trips are necessary, and whether they are included in the ETE analysis. In addition, the County RERPs indicate that the majority of school children do not have onsite bus transportation. Please describe the school bus mobilization time, and explain how this statement is consistent with the ETE statement that school evacuation resources would be immediately available.

Response to 13.3-15j

As explained previously, the composite traffic loading curve used for the ETE analysis is based on the data collected during evacuations executed in response to large scale chemical spills, and explicitly incorporates the time required for communication of the warning (warning diffusion) and the time required for an individual to respond to the warning (mobilization). The data collected in the meta-study was based on transient, permanent, and special populations and is therefore appropriate to use as “general” warning diffusion and public mobilization curves for all three population types. Therefore, the evacuation of school children is implicitly represented in this composite traffic loading curve.

County RERPs and the Nuclear Emergency Information Calendars do not contain information on trip generation times for the schools, and return trips for evacuating school children.

Based on the information available in county RERPs and the Nuclear Emergency Information Calendar, it can be concluded the plans and resources are in place to

evacuate school children. Some of the information found in county RERPs and Nuclear Emergency Information Calendar is provided below.

Louisa County RERP and Spotsylvania County RERP identify schools under special needs individuals group. In the “Organization” section of the Louisa County RERP, the superintendent of schools will

- Provide buses and other vehicles with drivers for assisting in an evacuation of the public and school students.
- Assist Orange County in operating a reception center designated as a host for Louisa County schools.

In the same section it is mentioned that the “Orange County School System will provide Gordon Barbour Elementary School as a reception center to be used for Louisa County schools.”

The School Evacuation Procedures section in the Nuclear Emergency Planning Information Calendar states that “If school children have not been returned to their homes before an evacuation order, they will be taken to an Evacuation Assembly Center (EAC) under adult supervision. You will be notified through your EAS radio or television stations if your children are being transported from their schools to one of the local Evacuation Assembly Centers. Make sure your family knows when and where to go. If you are separated from your family and have registered at an Evacuation Assembly Center, a record of your location will be available so that families may reunite.”

Schools can typically be expected to respond significantly faster than the general population. As a result of routine fire drills and other emergency drills performed at schools, response times are typically much better for school populations than for general populations. During school evacuation drills conducted as part of annual exercises within the Chemical Stockpile Emergency Preparedness Program (CSEPP), schools have demonstrated the ability to load buses and start evacuation within 10–20 minutes following a warning and directions to do so from local emergency management.

The analysis in the Evacuation Time Estimates for the North Anna Power Station and Surrounding Jurisdictions (2001) report was based on the availability of these resources.

RAI 13.3-15k

> Please describe whether the ETE provides for working people to return home to evacuate as a family unit. Also, please describe whether the ETE accounts for packing, closing up the home for extended evacuation, pickup of neighbors, businesses securing assembly lines, and farmers feeding or watering livestock prior to leaving. In addition, describe whether trip generation times have been considered for the agricultural and ranching operations identified in the County RERPs.

Response to 13.3-15k

The composite traffic loading curve used for the ETE incorporates the time required for communication of the warning to the public (warning diffusion time) and the time required for an individual to respond to the warning once received (mobilization time). The mobilization time distribution is based on data from actual emergency evacuations, and hence does implicitly account for most of the common activities performed by individuals in preparation for evacuation. However, the composite traffic loading curve does not explicitly account for activities such as returning home to evacuate as a family, packing, closing up the home for extended evacuation, pickup of neighbors, businesses securing assembly lines, and farmers feeding or watering livestock prior to leaving. Trip times for agricultural and ranching operations were not considered separately from the general population.

RAI 13.3-15l

> Please provide a figure (map) showing only those roads used as primary evacuation routes (e.g., Figure 3 of Appendix 4 to NUREG-0654/FEMA-REP-1), and also indicating the sector and quadrant boundaries.

Response to 13.3-15l

Figure 1 (at end of this response) shows the primary evacuation routes, the sector and quadrant boundaries.

RAI 13.3-15m

Please provide a figure (map) showing both the protective action zones (PAZs) and 10-mile emergency planning zone (EPZ) sector and quadrant boundaries.

Response to 13.3-15

Figure 2 (at end of this response) shows both the protective action zones (PAZs) and10-mile emergency planning zone (EPZ) section and quadrant boundaries.

RAI 13.3-15n

Please explain the basis for assuming that peak season night time evacuation represents the worst case scenario.

Response to 13.3-15n

The area surrounding the NAPS is extremely light in industry (as noted in Table 6 of the Evacuation Time Estimates for the North Anna Power Station and Surrounding Jurisdictions (2001) report, there are only four businesses other than NAPS that employ more than 50 employees). The area's population in the daytime is reduced greatly due to the population egress for work. The peak season night time scenario is the scenario that would place the largest number of cars on the evacuation routes, and is therefore deemed the worst case scenario.

RAI 13.3-15o

Please provide a figure (map) showing evacuation areas, shelter areas, and relocation centers in host areas.

Response to 13.3-15o

Evacuation and shelter areas are the same as the PAZs that are shown in Figure 2. Relocation centers are the same as Evacuation Assembly Centers (EACs) that are shown in Figure 3 (at end of this response).

RAI 13.3-15p

ETE Section 3.1.2 states that a car occupancy factor of 2.5 is assumed. Please provide justification and site-specific data for this number, as it applies to residents, transients, tourists, industries, and working people.

Response to 13.3-15p

The average household size for the five counties surrounding the NAPS is 2.56 (see Table 3 of the Evacuation Time Estimates for the North Anna Power Station and Surrounding Jurisdictions (2001) report). It is assumed that families would evacuate together in most situations. Additionally, Rogers notes that “most planners estimate that to evacuate, each household will take an average of 1.3 to 1.5 vehicles (or about 2 to 2.5 people per vehicle). These estimates have been derived primarily from evacuation intent surveys” (reference: 1990 ORNL study, "Evaluating Protective Actions for Chemical Agent Emergencies.") The U.S. Nuclear Regulatory Commission has also noted that “a range of two to three persons per vehicle would probably be reasonable in most cases” (reference: NUREG-0654/FEMA-REP-1, Rev.1, Criteria for Preparation and Evaluation of Radiological Emergency Response Plans and Preparedness in Support of Nuclear Power Plants, 1980). Transients, special facility, and workforce populations are also assumed to act similarly to the permanent resident population in terms of average car occupancy.

RAI 13.3-15q

ETE Section 3.1.2 states that an evacuation is deemed complete when 90% of the affected population (all of those evacuating) have exited the 10-mile EPZ. Neither the COVRERP nor County RERPs indicate that the ETE is for 90% of the population instead of 100%. In addition, the ETE Executive Summary implies that the total 10-mile EPZ population of 20,292 is included in the time estimates, instead of the actual total of 18,782 shown in ETE Table 9 (Summary of Results of Evacuation Time Analysis). Please explain how these assumptions are consistent.

Response to 13.3-15q

As mentioned in Section 3.1.2 of the Evacuation Time Estimates for the North Anna Power Station and Surrounding Jurisdictions (2001) report, calculating when 90% of the affected population (all of those evacuating) exits the 10-mile EPZ is typical for evacuation analyses and serves to prevent the long tail of the distribution (resulting from a small segment of the population being non-compliant, either initially or totally) from artificially inflating ETEs.

As footnote of the Table 9 states "the total population evacuated at the end of evacuation simulations to calculate ETEs is *approximately* 90% of the population loaded onto the evacuation network during the simulation". The population figure of 18,782 in the table is approximately 92.5% of the total 10-mile EPZ population of 20,292. The data presented in the table is the number of people evacuated, and not the total population. This is due to an artifact of the resolution of times available from the ESIM model. The total population evacuated presented in the Table 9 is not *exactly* 90% but *approximately (close to)* 90% of the total population.

RAI 13.3-15r

> ETE Section 4.5 (Confirmation of Evacuation) states that the most time-consuming method to confirm evacuation is to use ground vehicles, and that the time depends on the driving time for each route selected. Please provide the time needed for confirmation of evacuation, including the supporting assumptions and data.

Response to 13.3-15r

As mentioned in the Section 4.5 of the Evacuation Time Estimates for the North Anna Power Station and Surrounding Jurisdictions (2001) report, the time needed for confirmation of evacuation depends on the method of confirmation employed. County RERPs and Nuclear Emergency Information Calendar do provide some information on conducting such confirmation. The most applicable sections from RERPs and Nuclear Emergency Information Calendar are provided below (also the response to the question 8):

The Louisa County Radiological Emergency Response Plan – Basic Plan states that "Those requiring transportation will be identified, before or at the time of the emergency, and transportation will be provided."

The Nuclear Emergency Planning Information Calendars distributed in these counties stated that "When you have been notified, put the yellow "We Have Been Notified" card (in the back of this calendar) in your window or door facing the street or road when you leave your home or business. If you cannot print the notice, tie a towel to your door or mailbox. This tells emergency workers going door-to-door that you know about the emergency."

The Nuclear Emergency Planning Information Calendars also have information for the elderly and disabled people. It states that "If you have a special need for emergency notification, or if elderly or disabled persons require special care, special transportation

or another form of assistance, contact the Emergency Management Coordinator/Official for your county or city now, so that arrangements can be made in advance. Put the orange *'Special Assistance Needed' card* (provided to applicable residents in an Emergency Planning calendar) in your window or doorway facing the street or road. This will signify to emergency workers that special assistance is required at your address."

RAI 13.3-15s

> Please provide the separate distributions functions for the different categories of the population, and for each of the action stages after notification (e.g., see Section IV.B and Figure 4 of Appendix 4 of NUREG-0654/FEMA-REP-1).

Response to 13.3-15s

The composite traffic loading curve used for the ETE analysis is based on the data collected during evacuations executed in response to large-scale chemical spills, and explicitly incorporates the time required for communication of the warning to the public (warning diffusion) and the time required for an individual to respond to the warning (mobilization). The data collected in the meta-study was based on transient, permanent, and special populations and is therefore appropriate to use as "general" warning diffusion and public mobilization time distributions for all three population types. IEM did not use separate distributions for each population category. The warning diffusion time distribution (based on the use of an emergency alert system and sirens), and the public mobilization time distribution used for the ETE analysis are presented in the Figures 4 and 5, respectively, of the Evacuation Time Estimates for the North Anna Power Station and Surrounding Jurisdictions (2001) report.

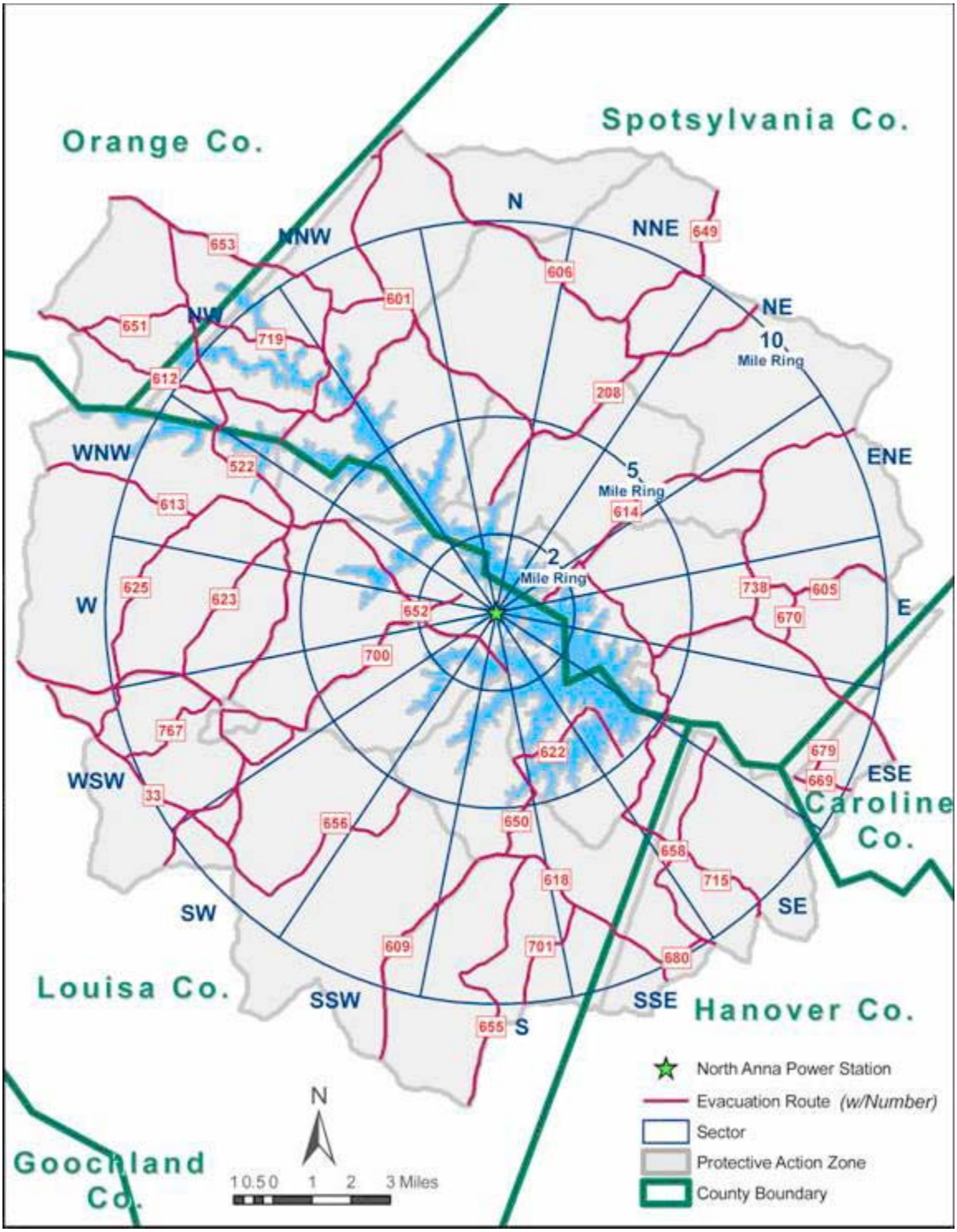


Figure 1: NAPS Primary Evacuation Routes—Sector and Quadrant Boundaries

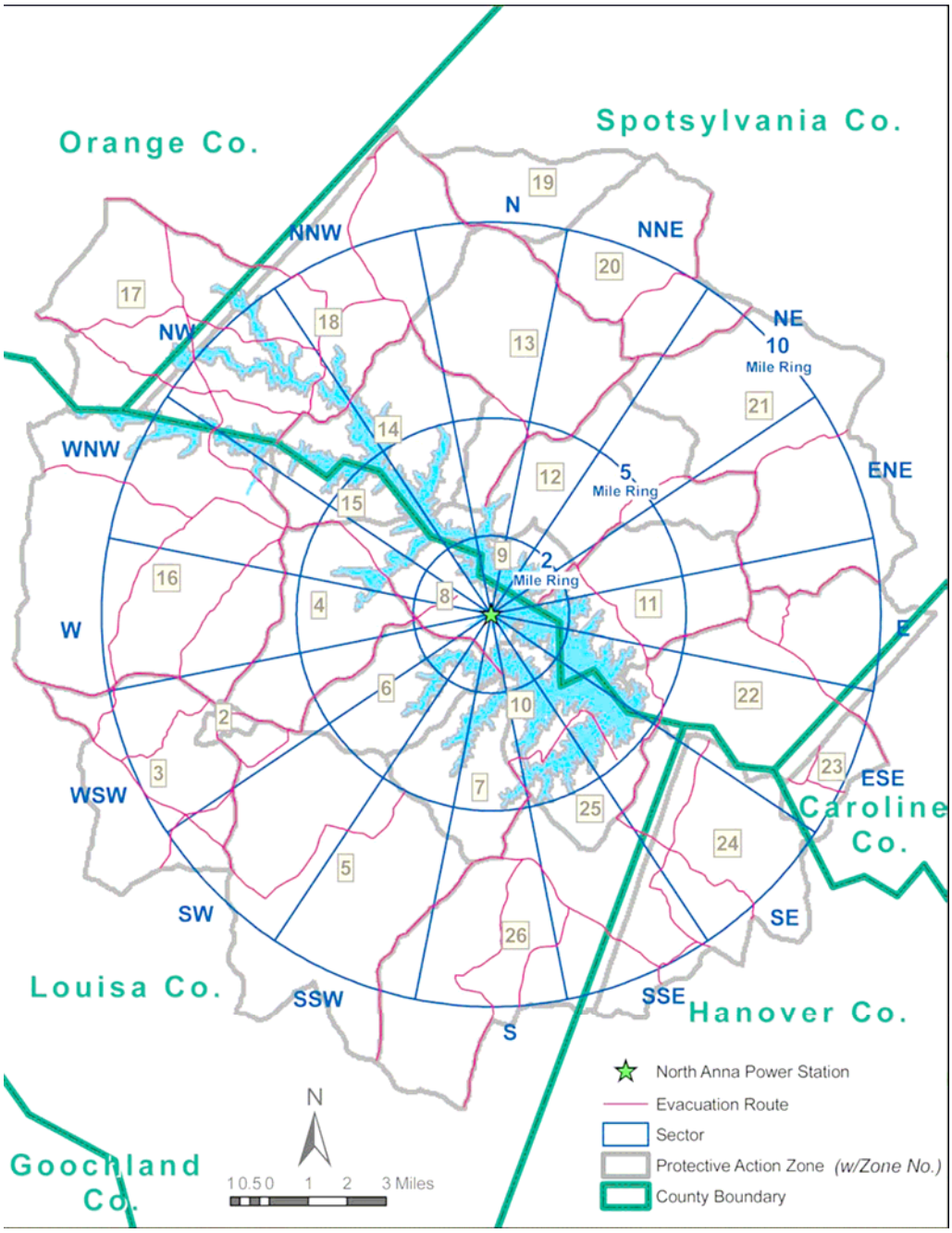


Figure 2: NAPS PAZs, Sector, and Quadrant Boundaries

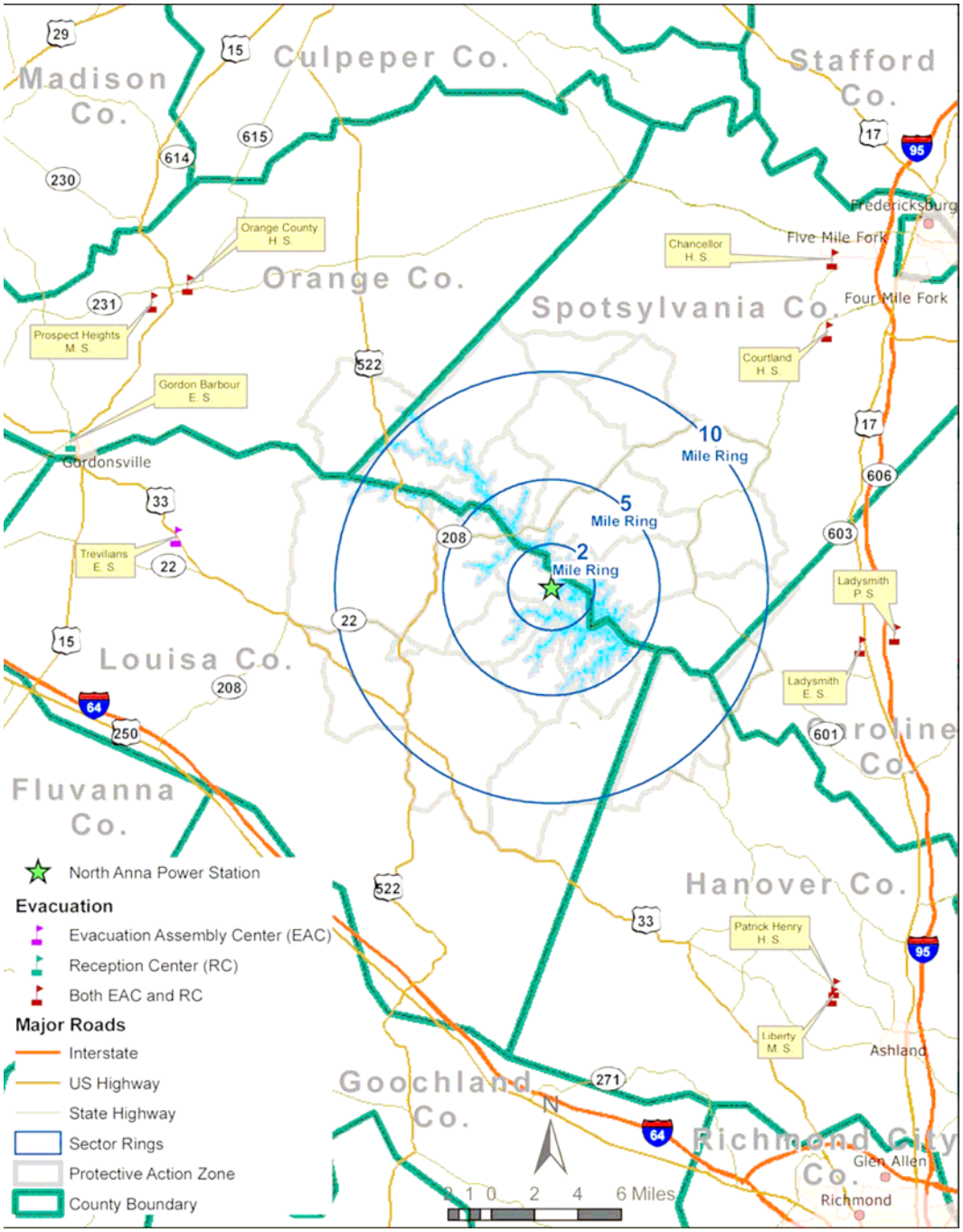


Figure 3: NAPS Evacuation Assembly Centers and Reception Centers