From: "Aughtman, Amy G." <AGAUGHTM@SOUTHERNCO.COM>
To: "Christian Araguas" <CJA2@nrc.gov>
Date: 2/13/2007 4:31:25 PM
Subject: Hydrology information needs-SNC response
cc: "Mark Notich" <mdn@nrc.gov>,"James T. Davis" <JTDAVIS@southernco.com>,"Tom C. Moorer" <TCMOORER@southernco.com>

Christian,
Please find attached SNC's response to four of the hydrology information needs (#s 4, 16, 19, and 23, not necessarily in that order and presented in this letter as items 1-4) discussed at the site audit.

Please contact me or Jim if you have any questions.
Thanks,
Amy A.

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SOUTHERN COMPANY
Energy to Serve Your World[SM]

FEB 13 2007

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

Southern Nuclear Operating Company
Vogtle Early Site Permit Application
Safety Review Site Audit Hydrology Information Needs

Ladies and Gentlemen:

On January 10, 2007, the U.S. Nuclear Regulatory Commission (NRC) performed a hydrology safety review audit at the Vogtle Electric Generating Plant (VEGP) site as part of their overall technical review of the Southern Nuclear Operating Company (SNC) Vogtle Early Site Permit (ESP) Application. Prior to the audit, the NRC provided SNC with a list of hydrology-related information needs required to support their review of the Site Safety Analysis Report (SSAR) portion of the ESP application. These information needs were discussed as part of the site audit and SNC agreed to provide formal responses to four of the information needs. It is SNC's understanding that the remaining hydrology information needs will be issued as formal requests for additional information (RAIs) by the NRC in the near future. In the enclosure to this letter, SNC is providing responses to the four hydrology information needs.

The SNC licensing contact for this information needs letter is J. T. Davis at (205) 992-7692.

Mr. J. A. (Buzz) Miller states he is a Vice President of Southern Nuclear Operating Company, is authorized to execute this oath on behalf of Southern Nuclear Operating Company and to the best of his knowledge and belief, the facts set forth in this letter are true.

Respectfully submitted,

SOUTHERN NUCLEAR OPERATING COMPANY

JAMiller

Joseph A. (Buzz) Miller

Sworn to and subscripted before me this 13th *day of* February*, 2007*

Gloria W. Buie
Notary Public

My commission expires: 05/06/08

JAM/BJS/dmw

Enclosure: Responses to NRC Site Hydrology Information Needs From January 2007 Safety Review Site Audit For the Vogtle ESP Application

cc: Southern Nuclear Operating Company
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Southern Nuclear Operating Company

AR-07-0302

Enclosure

Responses to

NRC Hydrology Information Needs

From

January 2007 Safety Review Site Audit

For the

Vogtle ESP Application

Information Needs from the January 2007 Safety Review Site Audit

SNC's responses to the Hydrology audit information needs (INs) are provided below. Where answers change facts and conclusions presented in the ESP application, the application will be revised. Responses that provide clarification detail will be considered for inclusion in the next revision as appropriate.

SSAR Section 2.4.4, Potential Dam Failures

Item #1 Present the calculations used to estimate the dam breach parameters discussed on page 2.4.4-4 and outlined in the USBR.

Response:

Breach Parameters

The HEC-RAS computer program dam breach option requires the input of several breach parameters. These include the final bottom width (B) and the bottom elevation of the breach along with the side slopes (Z) of the breach. The time (tf) to reach the final breach dimensions is also required input. Several methodologies are available to estimate these parameters. The U.S. Bureau of Reclamation (USBR) has summarized many of these methodologies in a single document, Prediction of Embankment Dam Breach Parameters (Reference 1). These methodologies give various results. The breach parameters for the Thurmond and Russell Dams are estimated using many of the procedures described in Reference 1 and the results compared.

The formulas used for each of the breach parameter estimation methods are shown in the Appendix to this response (see page 8 of enclosure). The input and output variables for each of these formulas are meters, cubic meters, and hours. Several variables for each of these methods are required. The required variables are listed below:

h_w = Depth of water at dam at failure, above the breach bottom, (m)
h_b = Height of breach (m)
h_d = Height of dam (m)
S = Storage volume at breach elevation (m^3)
S* = Dimensionless storage, (S/h_b^3)
Wc = Width of dam crest (m)
Wb = Width of dam bottom (m)
W* = Dimensionless average dam width, ($(Wc + Wd)/2h_b$)
Ver = Volume of material eroded, Estimated by ($0.0261(S^*h_w)^{0.769}$), (m^3)
Ko = Overtopping correction factor (1.4 if failure mode is overtopping)
Kc = Core wall correction factor (0.6 if dam has a core wall, 1.0 if not)

The breach for each dam will consist of an overtopping breach. The breach depth for each dam is also estimated to reach to the upstream reservoir invert. For both the Russell and Thurmond Dams, this is a conservative estimate since the majority of the portions of each dam that reach the upstream inverts are the portions constructed of concrete where the tainter gate spillways and hydroelectric turbines are located. In order for the earth sections to breach to the invert depth for the widths calculated below, native material will have to be eroded. However, for the purpose of this analysis, it will be assumed that the embankment and native material will erode to the upstream invert elevation.

The input variables along with the estimated breach parameters, by the various methodologies, for each dam are shown below:

J. Strom Thurmond Dam Breach Parameters

Input Variables

Input Variable	English Units	SI Units
h_w	151.1 ft	46.1 m
h_b	151.0 ft	46.0 m
h_d	151.0 ft	46.0 m
S	4360000 ac-ft	5378009947 m^3
S*		55162.75
Wc	40 ft	12.2 m
Wb	740 ft	225.6 m
W*		8.47
Ver		15085176.57 m^3
Ko		1.4
Kc		0.6

Breach Parameters

Reference	B (m)	B (ft)	Z	tf (hrs)
Johnson and Illes	138.1	453		
Singh and Snorrason (1982, 1984)	230.1	755		0.25 to 1.0
MacDonald and Langridge-Monopolis (1984)				7.34
FERC (1987)	230.1	755	1 to 2	0.1 to 1.0
Froehlich (1987)	365.6	1199	2.1	
Bureau of Reclamation (1988)	138.2	453		1.52
Von Thun and Gillette	170.0	558		1.17
Froehlich (1995b)	679.0	2228		11.62

Guidelines from FERC (1987) as well as other sources in the literature indicate that the breach width should be 2 to 5 times the height of the dam. This guidance is confirmed by the USBR (1998) report (Reference 1, Figure 9), which shows the 84 data points for observed breach widths used in their analysis of dam breach parameters. The Froehlich (1995b) relationships were developed using a regression analysis of the data, which is biased by the fact that the majority of the data points are for breach widths less than 50 meters. In fact, the USBR (1998) states that the Froehlich relationships are apparently the best fit for cases with observed breach widths less than 50 meters. Extrapolation of the Froehlich relations to the anticipated breach width on the order of 5 times the height of the dam (230 meters) indicates that the Froehlich relations are not in agreement with the observed data for breach widths greater than 150 meters. Since all of the other methods shown in the breach parameters table above are of the same order of magnitude, and are also within the range of accepted engineering practice for FERC-mandated dambreak analyses, a breach width of 755 feet was selected for this study. The value of 755 feet also is the maximum of the values obtained by all other methods, and is therefore conservative. The following considerations of the dam layout and river cross-section at the dam show that the use of a 755-foot breach width is also conservative in light of the physical layout of J. Strom Thurmond dam and appurtenances:

1) The HEC-RAS dam breach model and the equations used to determine discharges from the breach assume a "flat" bottom breach with a constant elevation. This means that bottom elevation of the entire 755-foot breach width is assumed to be at El. 200 ft, which is the minimum elevation of the original streambed on the upstream side of J Strom Thurmond Dam.

2) As shown on Item #1 Figure 1 of this response, the total dam width at the top of the dam is about 5,700 ft. (Reference 2). The width of the dam at the upstream invert elevation (El. 200 ft) is about 2,840 ft. Located within the portion of the dam that extends to El.. 200 ft is a concrete embankment section 2,282 ft wide where the tainter gate spillways and powerhouse are located (Reference 2). The failure mode assumes that only the earth section of the dam will erode during the breach. Consequently, the 755-foot bottom width of the breach extends beyond the area in which the actual ground elevation is at the minimum ground elevation of El. 200 ft.

3) Superposing the 755-foot bottom width at El 200 ft on the cross-section of the valley on the upstream side of the dam shows that more than 200 feet of the breach would be above El. 200 ft. Therefore, the entire bottom of the breach was taken as El. 200 ft to be conservative. The cross section shown in Figure 1 has been artificially widened at El. 200 ft to accommodate the 755 ft wide breach.

Based on a review of data and analyses for 84 dam failure cases, and the physical layout of J Strom Thurmond Dam, a breach width of 755 ft, with 2 to 1 side slopes was selected for this analysis. Additionally, most of the breach time predictions are close to 1.0 hour. Thus, a breach time of 1.0 hour was selected for this analysis.

B = 755 ft
Z = 2
tf = 1.0 hr.

Richard B. Russell Dam Breach Parameters

Input Variables

Input Variable	English Units	SI Units
Hw	150.1 ft	45.8 m
Hb	150.0 ft	45.7 m
Hd	150.0 ft	45.7 m
Storage	1700000 ac-ft	2096930484 m^3
S*		21941.45
Wc	20 ft	6.1 m
Wb	865 ft	263.7 m
W*		9.68
Ver		7274160.639 m^3
Ko		1.4
Kc		0.6

Breach Parameters

Reference	B (m)	B (ft)	Z	tf(hrs)
Johnson and Illes	137.2	450		
Singh and Snorrason (1982, 1984)	228.6	750		0.25 to 1.0
MacDonald and Langridge-Monopolis (1984)				5.63
FERC (1987)	228.6	750	1 to 2	0.1 to 1.0
Froehlich (1987)	258.3	847	2.4	
Bureau of Reclamation (1988)	137.3	450		1.51
Von Thun and Gillette	169.3	555		1.17
Froehlich (1995b)	501.7	1646		7.10

The breach width for the Richard B. Russell dam is also much larger than 50 meters and thus, the Froehlich equations predict values much greater than the observed data. Since all of the other methods shown in the breach parameters table above are of the same order of magnitude, and are also within the range of accepted engineering practice for FERC-mandated dambreak analyses, a breach width of 750 feet was selected for this study. The value of 750 feet also is the maximum of the values obtained by all other methods, and is therefore conservative. The following considerations of the dam layout and river cross-section at the dam show that the use of a 750-foot breach width is also conservative in light of the physical layout of Richard B. Russell dam and appurtenances:

1) The HEC-RAS dam breach model and the equations used to determine discharges from the breach assume a "flat" bottom breach with a constant elevation. This means that bottom elevation of the entire 750-foot breach width is assumed to be at El. 345 ft, which is the minimum elevation of the original streambed on the upstream side of Richard B. Russell Dam.

2) As shown on Item #1 Figure 2 of this response, the total dam width at the top of the dam is about 4,500 ft. (Reference 2). The width of the dam at the upstream invert elevation (El. 345 ft) is about 2,200 ft. Located within the portion of the dam that extends to El. 345 ft is a concrete embankment section 2,180 ft wide where the tainter gate spillways and powerhouse are located

(Reference 2). Only 1,000 ft of the concrete section extends to El. 345 ft, the remaining portion extends up the embankment. The failure mode assumes that only the earth section of the dam will erode during the breach. Consequently, the 750-foot bottom width of the breach extends beyond the area in which the actual ground elevation is at the minimum ground elevation of El. 345 ft.

3) Superposing the 750-foot bottom width at El. 345 ft on the cross-section of the valley on the upstream side of the dam shows that more than 150 feet of the breach would be above El. 345 ft. Therefore, the entire bottom of the breach was taken as El. 345 ft to be conservative. The cross section shown in Figure 2 has been artificially widened at El. 345 ft to accommodate the 750 ft wide breach.

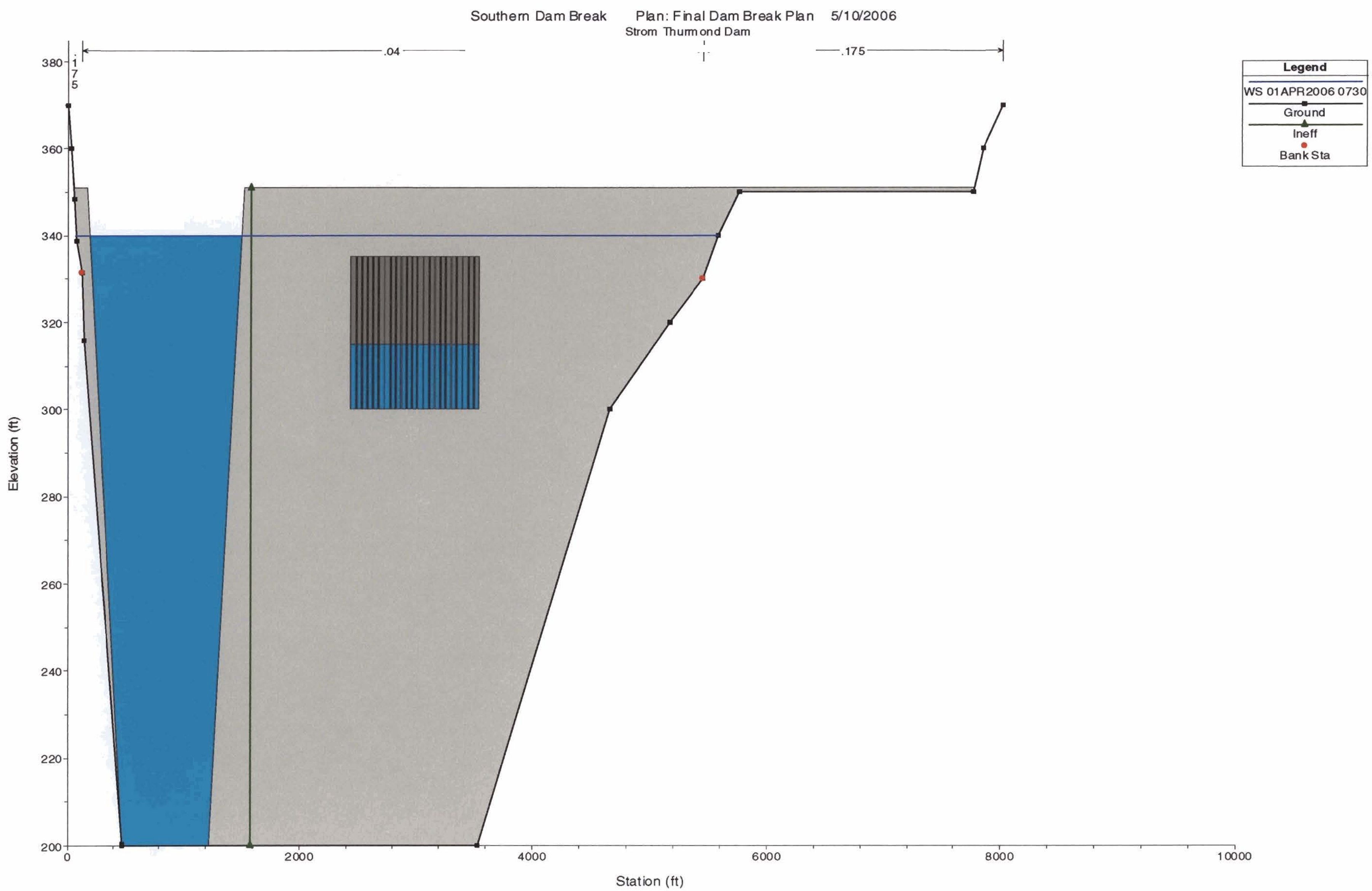
Based on a review of data and analyses for 84 dam failure cases, and the physical layout of Richard B. Russell Dam, a breach width of 750 ft, with 2 to 1 side slopes was selected for this analysis. Additionally, most of the breach time predictions are close to 1.0 hour. Thus, a breach time of 1.0 hour was selected for this analysis.

B = 750 ft
Z = 2
tf = 1.0 hr.

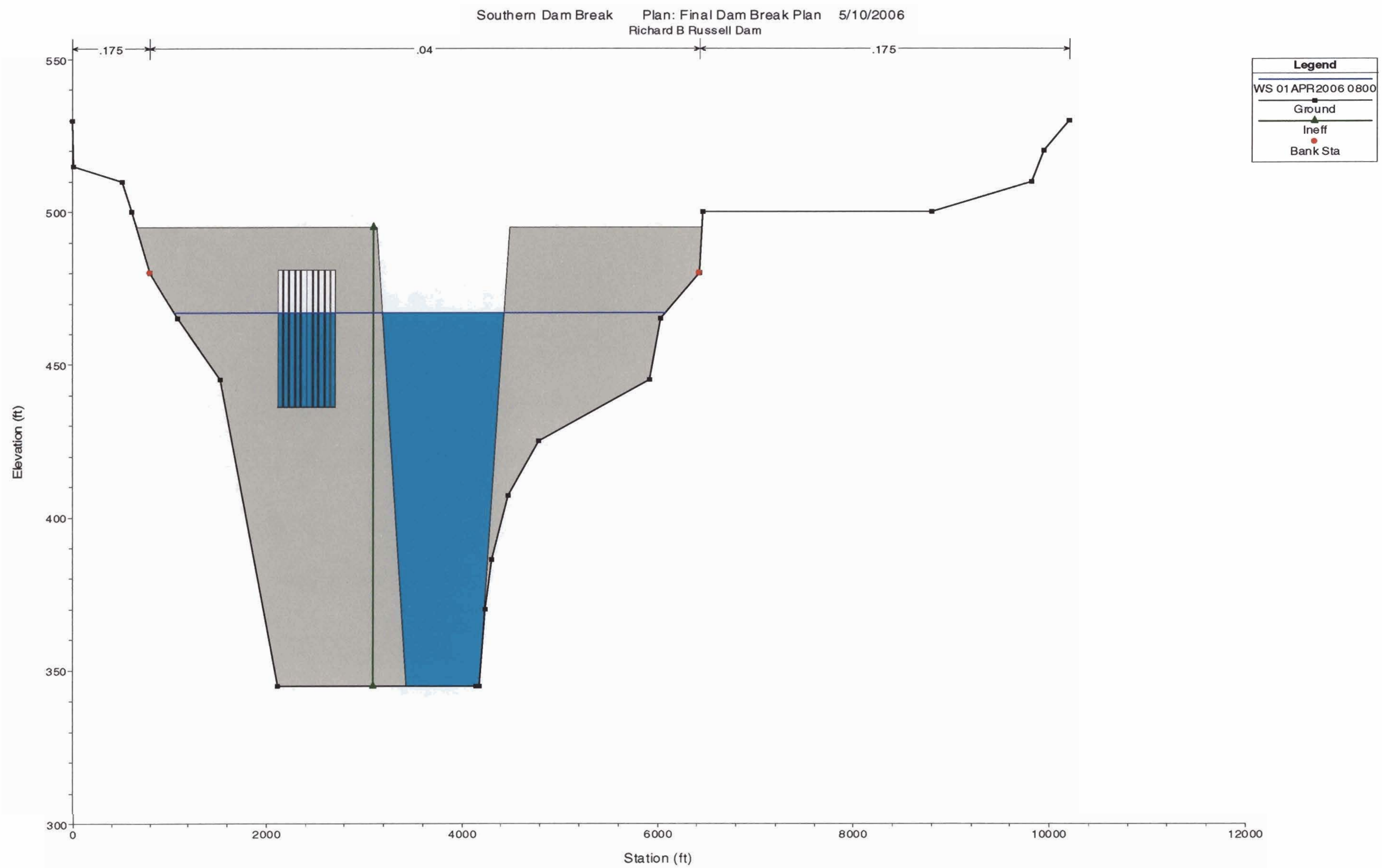
<u>Response References:</u>

1. Bureau of Reclamation, Water Resources Research Laboratory, Dam Safety Office, Dam Safety Research Report, DSO-98-004, Prediction of Embankment Dam Breach Parameters, July 1998.

2. U. S. Army Corps of Engineers, Savannah District, Savannah River Basin, Hartwell, Richard B. Russell, and J. Strom Thurmond Projects, Water Control Manual, 1996.



Item #1 Figure 1



Item #1 Figure 2

generally only qualititative or visual in nature. The digital image database is especially interesting.

Predicting Breach Parameters from Case Study Data

Table 2 summarizes the relations proposed by previous investigators for predicting breach parameters (e.g., geometry, time of formation) from case study data. The earliest contributions were made by Johnson and Illes (1976), who published a classification of failure shapes for earth, gravity, and arch dams. For earth dams, the breach shape was described as varying from triangular to trapezoidal as the breach progressed. The great majority of earth dam breaches are described as trapezoidal in the literature.

Table 2. — Breach parameter relations based on dam-failure case studies.
For explanations of symbols see the *Notation* section at the end of this report.

Reference	Number of Case Studies	Relations Proposed (S.I. units, meters, m^3/s, hours)
Johnson and Illes (1976)		$0.5h_d \le B \le 3h_d$ for earthfill dams
Singh and Snorrason (1982, 1984)	20	$2h_d \le B \le 5h_d$ 0.15 m $\le d_{ovtop} \le$ 0.61 m 0.25 hr $\le t_f \le$ 1.0 hr
MacDonald and Langridge-Monopolis (1984)	42	Earthfill dams: $V_{er} = 0.0261(V_{out}*h_w)^{0.769}$ [best-fit] $t_f = 0.0179(V_{er})^{0.364}$ [upper envelope] Non-earthfill dams: $V_{er} = 0.00348(V_{out}*h_w)^{0.852}$ [best fit]
FERC (1987)		B is normally 2-4 times h_d B can range from 1-5 times h_d Z = 0.25 to 1.0 [engineered, compacted dams] Z = 1 to 2 [non-engineered, slag or refuse dams] t_f = 0.1-1 hours [engineered, compacted earth dam] t_f = 0.1-0.5 hours [non-engineered, poorly compacted]
Froehlich (1987)	43	$\overline{B}^* = 0.47K_o(S^*)^{0.25}$ K_o = 1.4 overtopping; 1.0 otherwise $Z = 0.75K_c(h_w^*)^{1.57}(\overline{W}^*)^{0.73}$ K_c = 0.6 with corewall; 1.0 without a corewall $t_f^* = 79(S^*)^{0.47}$
Reclamation (1988)		$B = (3)h_w$ $t_f = (0.011)B$
Singh and Scarlatos (1988)	52	Breach geometry and time of failure tendencies B_{top}/B_{bottom} averages 1.29
Von Thun and Gillette (1990)	57	B, Z, t_f guidance (see discussion)
Dewey and Gillette (1993)	57	Breach initiation model; B, Z, t_f guidance
Froehlich (1995b)	63	$\overline{B} = 0.1803K_oV_w^{0.32}h_b^{0.19}$ $t_f = 0.00254V_w^{0.53}h_b^{(-0.90)}$ K_o= 1.4 for overtopping; 1.0 otherwise

Singh and Snorrason (1982) provided the first quantitative guidance on breach width. They plotted breach width versus dam height for 20 dam failures and found that breach width was generally between 2 and 5 times the dam height. The failure time, from inception to completion of breach, was generally 15 minutes to 1 hour. They also found that for overtopping failures, the maximum overtopping depth prior to failure ranged from 0.15 to 0.61 meters (0.5 to 2.0 ft).

MacDonald and Langridge-Monopolis (1984) proposed a breach formation factor, defined as the product of the volume of breach outflow (including initial storage and concurrent inflow) and the depth of water above the breach invert at the time of failure. They related the volume of embankment material removed to this factor for both earthfill and non-earthfill dams (e.g., rockfill, or earthfill with erosion-resistant core). Further, they concluded from analysis of the 42 case studies cited in their paper that the breach side slopes could be assumed to be 1h:2v in most cases; the breach shape was triangular or trapezoidal, depending on whether the breach reached the base of the dam. An envelope curve for the breach formation time as a function of the volume of eroded material was also presented for earthfill dams; for non-earthfill dams the time to failure was unpredictable, perhaps because, in some cases, failure may have been caused by structural instabilities rather than progressive erosion. The authors described iterative procedures for estimating breach parameters, simulating breach outflows using DAMBRK or other models, and revising breach parameter estimates as necessary.

Froehlich (1987) developed nondimensional prediction equations for estimating average breach width, average side-slope factor, and breach formation time. The predictions were based on characteristics of the dam, including reservoir volume, height of water above the breach bottom, height of breach, width of the embankment at the dam crest and breach bottom, and coefficients that account for overtopping vs. non-overtopping failures and the presence or absence of a corewall. Froehlich also concluded that, all other factors being equal, breaches caused by overtopping are wider and erode laterally at a faster rate than breaches caused by other means.

Froehlich revisited his 1987 analysis in a 1995 paper, using data from a total of 63 case studies. Eighteen of these failures had not been previously documented in the literature reviewed for this report. Froehlich developed new prediction equations for average breach width and time of failure. In contrast to his 1987 relations, the new equations are not dimensionless. Both 1995 relations had better coefficients of determination than did the 1987 relations, although the difference for the time of failure relation was very slight. Froehlich did not suggest a prediction equation for the average breach side slopes in his 1995 paper, but simply suggested assuming breach side slope factors of $Z = 1.4$ for overtopping failures or $Z = 0.9$ for other failure modes. He noted that the average side slope factor for the 63 case studies was nearly 1.0. The data set showed that there are some significant outliers in this regard.

Reclamation (1988) provided guidance for selecting ultimate breach width and time of failure to be used in hazard classification studies using the SMPDBK model. The suggested values are not intended to yield accurate predictions of peak breach outflows, but rather are intended to produce conservative, upper bound values that will introduce a factor of safety into the hazard classification procedure. For earthen dams, the recommended breach width is 3 times the breach depth, measured from the initial reservoir water level to the breach bottom elevation (usually assumed to be the streambed elevation at the toe of the dam). The recommended time for the breach to develop (hours) is 0.011 times the breach width (meters).

Singh and Scarlatos (1988) documented breach geometry characteristics and time of failure tendencies from a survey of 52 case studies. They found that the ratio of top and bottom breach widths, B_{top}/B_{bottom}, ranged from 1.06 to 1.74, with an average value of 1.29 and standard deviation of 0.180. The ratio of the top breach width to dam height was widely scattered. The breach side slopes were inclined 10-50° from vertical in most cases. Also, most failure times were less than 3 hours, and 50 percent of the failure times were less than 1.5 hours.

Von Thun and Gillette (1990) and Dewey and Gillette (1993) used the data from Froehlich (1987) and MacDonald and Langridge-Monopolis (1984) to develop guidance for estimating breach side slopes, breach width at mid-height, and time to failure. They proposed that breach side slopes be assumed to be 1:1 except for dams with cohesive shells or very wide cohesive cores, where slopes of 1:2 or 1:3 (h:v) may be more appropriate.

Von Thun and Gillette proposed the following relationship for average breach width:

$$\bar{B} = 2.5h_w + C_b \tag{1}$$

with h_w being the depth of water at the dam at the time of failure, and C_b a function of reservoir storage as follows:

Reservoir Size, m^3	C_b, meters	Reservoir Size, acre-feet	C_b, feet
< $1.23*10^6$	6.1	< 1,000	20
$1.23*10^6$ - $6.17*10^6$	18.3	1,000-5,000	60
$6.17*10^6$ - $1.23*10^7$	42.7	5,000-10,000	140
> $1.23*10^7$	54.9	>10,000	180

They noted that this relationship more accurately fits the full range of historical case study data than do the eroded embankment volume relations based on the breach formation factor proposed by MacDonald and Langridge-Monopolis. The volume of eroded embankment is useful, however, as a check on the reasonableness of breach geometries predicted by other means. Von Thun and Gillette presented a plot of eroded embankment volume versus water outflow volume and the depth of water above the breach invert, with contours indicating upper bounds of reasonable breach geometry estimates. They also noted that the small database of large-dam failures tends to indicate 150 meters (500 ft) as a possible upper bound for breach width.

Von Thun and Gillette proposed two methods for estimating breach formation time. Plots of breach formation time versus depth of water above the breach invert suggested upper and lower bound prediction equations for erosion resistant and easily eroded materials of:

$$t_f = 0.020h_w + 0.25 \qquad \textit{[erosion resistant]} \tag{2}$$

$$t_f = 0.015h_w \qquad \textit{[easily erodible]} \tag{3}$$

where t_f is in hours and h_w is in meters.

Von Thun and Gillette also developed equations for breach formation time based on observations of average lateral erosion rates (the ratio of final breach width to breach formation time) versus depth of water above the breach invert. They found a stronger correlation between the lateral erosion rate and depth than for the total breach formation time versus depth. Tests of fuse plug embankments intended to erode easily suggest upper bounds on the lateral erosion rate. Using lateral erosion rate data, Von Thun and Gillette put forth two additional equations:

$$t_f = \frac{\overline{B}}{4h_w} \qquad \textit{[erosion resistant]} \qquad (4)$$

$$t_f = \frac{\overline{B}}{4h_w + 61.0} \qquad \textit{[highly erodible]} \qquad (5)$$

with h_w and $\overline{B}$ both given in meters. Each of these equations requires an assumption or prediction of the average breach width.

These equations reflect both case study data and results of controlled laboratory tests of fuse plug embankments (Pugh, 1985) using both highly erodible and slightly cohesive materials.

Predicting Peak Outflows from Case Study Data

In lieu of determining breach parameters and then routing inflow and reservoir storage through the breach, many investigators have used the case study data to develop empirical equations relating peak breach outflow to dam height, reservoir storage volume, or combinations of the two. These relations are summarized in Table 1 and discussed in more detail below. Figures 13 through 15 also graphically show these relations compared to the case study data.

Kirkpatrick (1977) presented data from 13 embankment dam failures and 6 additional hypothetical failures, and proposed a best-fit relation for peak discharge as a function of the depth of water behind the dam at failure. This analysis included data from the failure of St. Francis Dam, California, which was a concrete gravity structure. St. Francis Dam was originally thought to have failed due to piping through the right abutment, but a recent study suggests that it may have failed due to a combination of overturning of a concrete gravity section and landslide failure of the left abutment, and thus may not be appropriate for inclusion in the analysis (Rogers and McMahon, 1993).

The Soil Conservation Service (1981) used the 13 case studies cited by Kirkpatrick to develop a power law equation relating the peak dam failure outflow to the depth of water at the dam at the time of failure. This appears to be an enveloping curve, although three data points are slightly above the curve. Reclamation (1982) extended this work and proposed a similar envelope equation for peak breach outflow using case study data from 21 dams.

NOTATION

β	Particle packing factor, ratio of roughness height to roughness spacing
B	Breach width (general)
$\overline{B}$	Average breach width $(B_{top} + B_{bottom}) / 2$
$\overline{B}^*$	Dimensionless average breach width $(\overline{B} / h_b)$
B_{top}	Breach width at top of breach
B_{bottom}	Breach width at bottom of breach
B_{avg}	Average breach width
C_b	Constant in Von Thun and Gillette breach width relation
c	Coefficient in equation for λ, dependent on aeration and particle packing factors
δ	Angle of repose
d	Drop in reservoir level through a breach (Walder and O'Connor, 1997)
d_m	Mean roughness height
d_{ovtop}	Depth of overtopping flow at failure
d_s	Equivalent stone diameter
D_c	Height of dam crest relative to dam base (Walder and O'Connor, 1997)
$\dot{\varepsilon}$	Erosion rate, mass/time
E_L	Lateral erosion rate, distance/time
f	Darcy's friction factor
g	Acceleration of gravity
γ_s	Unit weight of solid material
γ_w	Unit weight of water
η	Dimensionless parameter relating breach erosion rate and reservoir size (Walder and O'Connor, 1997)
h_b	Height of breach
h_d	Height of dam
h_w	Hydraulic depth of water at dam at failure, above breach bottom
h_w^*	Dimensionless height of water above breach bottom, (h_w / h_b)
J_a	Joint alteration number
J_n	Joint set number
J_r	Joint roughness number
J_s	Relative ground structure number
K_c	Core wall correction factor (0.6 if dam contains a core wall; 1.0 otherwise)
k	Mean vertical erosion rate of breach (Walder and O'Connor, 1997)
k_d	Erosion detachment rate coefficient
K_h	Headcut erodibility index
K_o	Overtopping correction factor (1.4 if failure mode is overtopping; 1.0 otherwise)
λ	Surface flow resistance factor (analogous to Darcy's f)
M_s	Earth mass strength number
Q_p	Peak breach outflow
Q_p^*	Dimensionless peak breach outflow, $Q_p/g^{1/2}d^{5/2}$, (Walder and O'Connor, 1997)
RQD	Rock quality designation
σ	Aeration factor, specific weight of air-water mixture divided by specific

	weight of pure water
S	Storage
S^*	Dimensionless storage, (S/h_b^3)
T	Shields parameter
τ_c	Critical shear stress
τ_e	Erosionally effective stress
t_f	Breach formation time, hours
t_f^*	Dimensionless breach formation time, $t_f / \sqrt{gh_b}$ (t_f, g, and h_b must be in units that produce a dimensionless t_f^*.)
θ	Downstream embankment slope angle
v	Flow velocity down embankment slope
v_c	Critical velocity to dislodge riprap particles
V_{er}	Volume of embankment material eroded
V_0, V_{out}	Volume of water discharged through breach (initial storage + inflow during failure)
V_w	Volume of water above breach invert elevation at time of breach
$\overline{W}^*$	Dimensionless average embankment width $(W_{crest}+W_{bottom})/(2h_b)$
y_m	Mean water depth normal to embankment slope
Z	Breach opening side slope factor (Z horizontal:1 vertical)
$Z_{e/u}$	Upstream embankment face slope factor
$Z_{e/d}$	Downstream embankment face slope factor

SSAR Section 2.4.12 Groundwater

Item #2 Discuss the PROCESS used to estimate the outflow to Mallard Pond (0.07 gpm).

Response:

The rate at which a release from an effluent holdup tank, as discussed in the response to Item #3, discharges to surface water (Mallard Pond) is determined by the transport characteristics of the water table aquifer. A release from an effluent holdup tank would undergo unsaturated zone transport beneath the auxiliary building, followed by saturated zone transport first through the backfill and then through the Utley limestone, and would finally discharge to Mallard Pond. The discharge rate itself is a function of the Darcy velocity and the assumed volume and dimensions of the resulting contaminant slug. The Darcy velocity was calculated to be 0.047 ft/day, using a hydraulic conductivity of 3.3 ft/day and a hydraulic gradient of 0.014 ft/ft. These values represent the hydrogeologic characteristics of the backfill as described in the response to Item #3. The volume of the liquid release has been assumed to be 22,400 gal. (2,995 ft^3), which represents 80 percent of the 28,000 gal. capacity of one effluent holdup tank (NUREG-0800, Section 15.7.3, assumes that 80 percent of the liquid volume is considered in this analysis). Considering the effective porosity of the backfill, given as 0.34 in the response to Item #3, the release would occupy about 8,810 ft^3 of the saturated backfill. The shape of the resulting contaminant slug is assumed to be square in plan view and extend vertically throughout the entire saturated thickness of the backfill. Using 20 ft as a representative saturated thickness (water table to top of Blue Bluff Marl), the slug would have an area of about 440 ft^2 in plan view and a width of about 21 ft. The cross-sectional area of the contaminant slug normal to the groundwater flow direction would therefore be 20 ft by 21 ft or about 420 ft^2. The discharge rate of the contaminant slug is then the product of the Darcy velocity and the cross-sectional area, 20 ft^3/day or 0.10 gpm.

(Note that the ESP application gives the discharge rate of a postulated release into Mallard Pond as 0.7 gpm. This value should have been reported as 0.07 gpm. The dilution factor of 2.8E-04 was calculated correctly as the ratio of 0.07 gpm to 250 gpm.)

Response References:

None.

SSAR Section 2.4.13 Accidental Releases of Liquid Effluents in Ground and Surface Waters

Item #3 Discuss the process used to establish that the conceptual model for the transport pathways and travel times presented in the SSAR represents the most conservative (pessimistic) pathway from various other feasible alternative pathways.

Response:

As described in SSAR Section 2.4.13 of the ESP application, the bounding accident scenario has been identified as an accidental release of liquid effluent from an effluent holdup tank located in the lowest level of the AP1000 auxiliary building. The ground and surface water pathways associated with this accident scenario include the following components: (1) vertically-downward, unsaturated zone transport from the base of the AP1000 auxiliary building to the water table; (2) predominantly horizontal, saturated zone transport in the unconfined aquifer from the point of release to Mallard Pond, a groundwater discharge area located about 2,200 ft north of the auxiliary buildings; (3) mixing of contaminated and uncontaminated water in Mallard Pond; (4) transport and mixing in the unnamed stream draining Mallard Pond and discharging to the Savannah River; and (5) mixing of contaminated and uncontaminated water in the Savannah River with subsequent transport downstream.

The conceptual model and associated pathways were established based on the AP1000 design, site-specific hydrogeological data for the VEGP site, and published data, including, but not limited to, the UFSAR for VEGP Units 1 and 2. The travel times for these pathways were estimated using site-specific data. In cases where the site data exhibited variability, the maximum or minimum value was selected that would lead to the most conservative (pessimistic) estimate of travel time. For some of the pathways, the travel time was conservatively taken to be zero and processes that would further attenuate radionuclide concentrations were conservatively ignored. The key conservative assumptions built into the transport analysis presented in SSAR Section 2.4.13 are identified and discussed below.

- The entire contents of a liquid waste effluent holdup tank were assumed to enter the water table aquifer instantaneously. This assumption is very conservative because it requires failure of the floor drain system, and it ignores the barriers presented by the 6-ft-thick base mat and the 3-ft-thick, sealed exterior walls of the auxiliary building. Furthermore, there would be a minimum of 20 ft of unsaturated zone beneath the base mat. Attenuation of radionuclide concentrations would occur during unsaturated zone transport as a consequence of adsorption, dispersion, and radioactive decay, which was not considered in this analysis.

  Radionuclide travel times in the water table aquifer were estimated using the most conservative of the hydraulic conductivity and distribution coefficient values characterized for the ESP site. Hydraulic conductivity values determined for the water table aquifer and reported in SSAR Table 2.4.12-3 range from 0.12 to 2.7 ft/day. A value of 2.7 ft/day was used in the analysis. Distribution coefficients characterized for the water table aquifer and reported in SSAR Table 2.4.13-3 range from 3.9 to 21.3 mL/g, 14.4 to 17.4 mL/g, and 22.7 to 33.2 mL/g for cobalt, strontium, and cesium, respectively. The lowest end of the range was used in the transport analysis. The use of the maximum hydraulic conductivity and minimum distribution coefficients underestimates the radionuclide travel times, which results in an underestimate of the attenuation due to radioactive decay. Furthermore, hydrodynamic dispersion during saturated zone transport was conservatively ignored. Hydrodynamic dispersion would further attenuate radionuclide concentrations due to the processes of mechanical dispersion and molecular diffusion.

Additional dilution downstream of Mallard Pond was not considered. Compliance with 10 CFR Part 20 is demonstrated if the failure of tanks and associated components containing radioactive liquids outside containment does not result in radionuclide concentrations in excess of the limits in 10 CFR Part 20, Appendix B, Table 2, Column 2, at the nearest potable water supply in an unrestricted area. NUREG-0800, Section 15.7.3 defines supply as a well or surface water intake that is used as a water source for direct human consumption or indirectly through animals, crops or food processing. Based on these requirements and definitions, the Savannah River is the nearest potable water supply in an unrestricted area. Therefore, compliance with 10 CFR Part 20 must be demonstrated in the Savannah River. Radionuclide concentrations in the stream leaving Mallard Pond would undergo significant dilution as the stream discharges to, and mixes with, the Savannah River.

Based on discussions with the NRC staff at the site audit and in order to incorporate additional conservatism in the hydrogeological properties of the water table aquifer, the transport analysis described in SSAR Section 2.4.13 has been re-performed. The calculation approach is the same, but it is performed using more conservative values for hydraulic conductivity and hydraulic gradient, and more representative values for the volume of liquid effluent released and dilution downstream of Mallard Pond. The assumptions and bases used to establish the transport parameters, and associated results are described below.

The groundwater travel time has been re-estimated by considering the actual locations of the effluent holdup tanks, the hydrogeologic properties of the backfill, and more conservative estimates of the hydraulic gradient and hydraulic conductivity of the water table aquifer. The total saturated zone travel time has been estimated as the sum of three components: (1) travel time in the backfill; (2) travel time in the water table aquifer in the area between the backfill and the point at which the hydraulic gradient steepens near OW-1005; and (3) travel time between OW-1005 and Mallard Pond. The travel time in each is a function of the travel distance, hydraulic conductivity, effective porosity, and hydraulic gradient. The basis for estimating the travel time in each of these three segments is described below.

1. The travel distance in the backfill was determined to be about 460 ft, which represents the shortest distance between the portion of level 1 of the auxiliary building potentially flooded by a tank rupture and the northern extent of the power block excavation. This distance considers the 71 ft between column lines 7.3 and 11 of the auxiliary building (AP1000 Doc. No. APP-1010-P2-001), the 310 ft length of the turbine building (AP1000 Doc. No. APP-0030-X4-001), and the 80 ft between the turbine building and the northern extent of the power block excavation. A hydraulic conductivity of 1,220 ft/yr (3.3 ft/day) was conservatively assigned to the backfill, which is the maximum in situ value measured for Units 1 and 2 (VEGP UFSAR, Table 2.4.12-15). The effective porosity of the backfill was taken to be 0.34 (VEGP UFSAR, Section 2.4.13.1.1). Because the backfill for Units 3 and 4 will be obtained from the borrow areas used for Units 1 and 2 and compacted to the same criteria, the hydraulic conductivity and porosity values observed for Units 1 and 2 should be representative of Units 3 and 4. The hydraulic gradient in the backfill was conservatively estimated to be 0.014 ft/ft using the maximum water level observed at OW-1009 (El. 163.03 ft msl), the minimum water level observed at OW-1005 (El. 132.53 ft msl), and the distance between the two observation wells (2209 ft). Based on the aforementioned, conservatively-established parameters, the groundwater travel time in the backfill was calculated to be 9.16 years.

2. The travel distance between the northern extent of the power block excavation and OW-1005 was determined to be 990 ft based on the location of OW-1005. Water table contour maps provided in the ESP application indicate that groundwater flows from the power block area to the north and towards Mallard Pond. Geotechnical borings in the area north of the Units 3 and 4 indicate that groundwater flow occurs in the Utley limestone, as the data suggest that the limestone is continuous along this pathway. Test results given in SSAR Table 2.4.12-3 indicate that the in situ hydraulic conductivity of the Utley limestone ranges from 0.12 to 2.7 ft/day (boring logs for wells OW-1003, OW-1005, OW-1006, OW-1007, OW-1009, OW-1010, OW-1013, and OW-1015 indicated completion in the Utley limestone). Hydraulic testing results reported in the UFSAR for adjacent VEGP Units 1 and 2 indicate the possibility of localized, highly permeable zones in the Utley limestone (see response to Item #4). To address the possibility that similar zones are present north of Units 3 and 4, the maximum value reported in the UFSAR, 125,400 ft/year (343 ft/day), is used in this re-analysis. The effective porosity of the water table aquifer has been estimated to be 0.32 based on site-specific measurements, as noted in SSAR Section 2.4.12.1.4. Effective porosities of limestone formations are typically lower. A value of 0.10 has been adopted from the literature (Heath 1998) to allow for the possibility that the effective porosity has been overestimated. The hydraulic gradient over this segment is assumed to be the same as that in the backfill (0.014 ft/ft). Using the parameters described above, a groundwater travel time of 0.06 years is estimated for this segment.

3. The travel distance between OW-1005 and Mallard Pond is about 1,000 ft based on site topographic surveys. As with the prior segment, groundwater flow occurs in the Utley limestone and the same values for hydraulic conductivity (125,400 ft/yr) and effective porosity (0.10) are adopted. The hydraulic gradient is estimated to be 0.023 ft/ft using the maximum water level observed at OW-1005 (133.20 ft msl), the water surface elevation in Mallard Pond (110 ft msl), and the distance between the two (1000 ft). A groundwater travel time of 0.03 years is estimated for this segment based on the above parameters.

Summing the above travel times, the total travel time for this conservative re-analysis is 9.25 years.

Following the approach in SSAR Section 2.4.13.1.3.1, transport considering radioactive decay only is assessed by decaying the liquid effluent assumed to be accidentally released for a period of 9.25 years. Results are summarized in Item #3 Table 1 of this response (comparable to SSAR Table 2.4.13-2) and indicate that H-3, Mn-54, Fe-55, Co-60, Sr-90, I-129, Cs-134, and Cs-137 would exceed their Maximum Permissible Concentration (MPC) by more than 1 percent.

Repeating the analysis described in SSAR Section 2.4.12.1.3.2, radionuclides exceeding their MPC by more than 1 percent (H-3, Mn-54, Fe-55, Co-60, Sr-90, I-129, Cs-134, and Cs-137) were further evaluated considering adsorption and retardation in addition to radioactive decay. Distribution coefficients for Co, Sr, and Cs were conservatively assigned using the site-specific results given by Kaplan and Millings (2006). For the backfill, Kaplan and Millings (2006) report distribution coefficients for 16 soil samples obtained from potential borrow source areas for the backfill (SSAR Table 2.4.13-3). To ensure conservatism, distribution coefficients representing the lower end of each range were chosen for radionuclide transport analysis (1.4 mL/g for Co, 6.0 mL/g for Sr, and 3.5 mL/g for Cs), all of which are associated with soil sample H-6. Distribution coefficients assigned to the Utley limestone are the same as those used previously (3.9 mL/g for Co, 14.4 mL/g for Sr, and 22.7 mL/g for Cs). Soil samples B-1003V-55-65, B-1003V-65-75, and B-1003V-75-82 were taken from a vibratory boring adjacent to boring B-1003. The boring log for B-1003 indicates that these samples represent Utley limestone.

Distribution coefficients for H-3 and I-129 were taken to be zero for both the backfill and the Utley limestone as was done previously. Distribution coefficients for Mn-54 and Fe-55 were conservatively assumed to be zero. Results of the transport analysis considering radioactive decay and adsorption are summarized in Item #3 Table 2 of this response (comparable to SSAR Table 2.4.13-4) and indicate that H-3, Mn-54, Fe-55, Sr-90, I-129, and Cs-137 would exceed their respective MPC by more than 1 percent.

Transport considering radioactive decay, adsorption, and dilution is assessed using the methodology described in SSAR 2.4.13.1.3.3. The dilution factor was, however, re-estimated to consider the more conservative assumptions regarding the hydrogeologic properties of the water table aquifer. The dilution factor is the ratio of the rate at which the postulated release would discharge to surface water (Mallard Pond) as contaminated groundwater to the total rate of groundwater discharge to surface water, which would include both uncontaminated and contaminated groundwater. The rate at which the release would discharge to Mallard Pond has been conservatively re-estimated to be 0.10 gpm (see response to Item #2). The rate of total groundwater discharge to surface water has been estimated as 1,125 gpm at a point just downstream of the confluence of the stream discharging from Mallard Pond and its west branch. This value is the result of stream flow measurements that were taken in the months of June and July to support the licensing of VEGP Units 1 and 2 (Bechtel 1985). Because the stream discharging from Mallard Pond and its west branch are both perennial streams, the stream flow measurements would represent the groundwater discharge. The resulting dilution factor is calculated as the ratio of 0.10 gpm to 1,125 gpm, or 9.1E-05. This dilution factor is applied to the H-3, Mn-54, Fe-55, Sr-90, I-129, and Cs-137 concentrations reported in Table 2 to account for dilution in addition to radioactive decay and adsorption. Item #3 Table 3 of this response summarizes the resulting concentrations, which would represent the concentrations in the surface water at a point just downstream of the confluence of the stream discharging from Mallard Pond and its west branch. It is seen that the concentrations of each of these radionuclides are below their respective MPCs.

As noted in SSAR Section 2.4.13.1.4, compliance with 10 CFR Part 20 imposes additional requirements when the identity and concentration of each radionuclide in a mixture are known. In particular, the ratio of concentration to MPC for each radionuclide present in the mixture must be determined, and the sum of such ratios for radionuclides in the mixture may not exceed unity. Following the methodology described in SSAR Section 2.4.13.1.4, the ratios were re-estimated using the more conservative assumptions described in this response. The results are provided in Item #3 Table 4 of this response (comparable to SSAR Table 2.4.13-5). The ratios for the mixture sum to 0.32, which demonstrates that an accidental liquid release of effluents in groundwater would not exceed 10 CFR Part 20 limits in the Mallard Pond stream, even for the more limiting and conservative analysis presented above.

Compliance with 10 CFR Part 20 is further assured considering that the point at which compliance has been demonstrated is within the restricted area and not a potable water source. The stream discharging from Mallard Pond is a gaining stream that discharges to, and mixes with, the Savannah River. The entire reach of this stream, about 1.0 mi. in length, is within the restricted area and not a potable water supply. The nearest potable water supply in an unrestricted area to which the 10 CFR Part 20 requirements would apply is the Savannah River. Mixing of the tributary stream flow with the Savannah River flow would dilute radionuclide concentrations further. The magnitude of this additional dilution can be estimated from the ratio of the tributary stream flow rate (1,125 gpm) to the Savannah River flow rate. Using the 100-year drought flow, given as 3,298 ft^3/sec (1,480,000 gpm) in SSAR Section 2.4.11, to conservatively represent the Savannah River flow rate, a dilution factor of 7.6E-04 is calculated. Accounting for this additional dilution would further reduce radionuclide concentrations by a factor of about 1,000.

Consequently, the ratios for the mixture would sum to a value much less than unity and well below the compliance limit.

Response References:

1. Bechtel, 1985, Flow Rate in Mathes Pond Stream & West Branch Stream, Calculation Number G-008, Vogtle Nuclear Power Plant, Job No. 9510-091.

2. Heath, R.C., 1998, Basic Ground-Water Hydrology, U.S. Geological Survey Water-Supply Paper 2220.

3. Javandel, I., Doughty, C. and Tsang, C-F, 1984, Groundwater Transport: Handbook of Mathematical Models, Water Resources Monograph 10, American Geophysical Union.

4. Kaplan, D.I., and Millings, M.R., 2006, Distribution Coefficients for the Vogtle Early Site Permit, WSRC-TR-2006-00246, Savannah River National Laboratory, Washington Savannah River Company, Aiken, South Carolina, July.

5. Kennedy, W.E., and Strenge, D.L., 1992, NUREG/CR-5512, Residual Radioactive Contamination From Decommissioning, Volume 1, Pacific Northwest Laboratory, October.

6. U. S. Department of Health, Education, and Welfare (USDOH), 1970, Radiological Health Handbook, January.

Item #3 Table 1 Results of Transport Analysis Considering Radioactive Decay Only

Radionuclide	Effluent Holdup Tank Concentration[1] ($\mu Ci/cm^3$)	Half-life[2] (days)	Decay Rate[3] ($days^{-1}$)	MPC[4] ($\mu Ci/cm^3$)	Groundwater Concentration[5] ($\mu Ci/cm^3$)	Groundwater Concentration/ MPC
H-3	1.01E+00	4.51E+03	1.54E-04	1.00E-03	6.01E-01	6.01E+02
Cr-51	1.31E-03	2.77E+01	2.50E-02	5.00E-04	2.57E-40	5.14E-37
Mn-54	6.77E-04	3.13E+02	2.21E-03	3.00E-05	3.82E-07	1.27E-02
Mn-56	1.72E-01	1.07E-01	6.48E+00	7.00E-05	0.00E+00	0.00E+00
Fe-55	5.05E-04	9.86E+02	7.03E-04	1.00E-04	4.70E-05	4.70E-01
Fe-59	1.31E-04	4.45E+01	1.56E-02	1.00E-05	1.85E-27	1.85E-22
Co-58	1.92E-03	7.08E+01	9.79E-03	2.00E-05	8.35E-18	4.18E-13
Co-60	2.22E-04	1.93E+03	3.59E-04	3.00E-06	6.60E-05	2.20E+01
Br-83	1.55E-02	9.96E-02	6.96E+00	9.00E-04	0.00E+00	0.00E+00
Br-84	8.24E-03	2.21E-02	3.14E+01	4.00E-04	0.00E+00	0.00E+00
Br-85	9.70E-04	2.01E-03	3.44E+02	1.00E+00	0.00E+00	0.00E+00
Rb-88	7.27E-01	1.24E-02	5.59E+01	4.00E-04	0.00E+00	0.00E+00
Rb-89	3.35E-02	1.06E-02	6.54E+01	9.00E-04	0.00E+00	0.00E+00
Sr-89	5.33E-04	5.05E+01	1.37E-02	8.00E-06	3.91E-24	4.89E-19
Sr-90	2.38E-05	1.06E+04	6.54E-05	5.00E-07	1.91E-05	3.82E+01
Sr-91	8.24E-04	3.96E-01	1.75E+00	2.00E-05	0.00E+00	0.00E+00
Sr-92	1.99E-04	1.13E-01	6.16E+00	4.00E-05	0.00E+00	0.00E+00
Y-90	6.30E-06	2.67E+00	2.60E-01	7.00E-06	0.00E+00	0.00E+00
Y-91m	4.46E-04	3.45E-02	2.01E+01	2.00E-03	0.00E+00	0.00E+00
Y-91	6.79E-05	5.85E+01	1.18E-02	8.00E-06	2.82E-22	3.53E-17
Y-92	1.65E-04	1.48E-01	4.68E+00	4.00E-05	0.00E+00	0.00E+00
Y-93	5.33E-05	4.21E-01	1.65E+00	2.00E-05	0.00E+00	0.00E+00
Nb-95	7.76E-05	3.52E+01	1.97E-02	3.00E-05	1.01E-33	3.36E-29
Zr-95	7.76E-05	6.40E+01	1.08E-02	2.00E-05	1.01E-20	5.03E-16
Mo-99	1.02E-01	2.75E+00	2.52E-01	2.00E-05	0.00E+00	0.00E+00
Tc-99m	9.70E-02	2.51E-01	2.76E+00	1.00E-03	0.00E+00	0.00E+00
Ru-103	6.79E-05	3.93E+01	1.76E-02	3.00E-05	9.11E-31	3.04E-26
Rh-103m	6.79E-05	3.90E-02	1.78E+01	6.00E-03	0.00E+00	0.00E+00
Rh-106	2.18E-05	4.63E-04	1.50E+03	NA[6]	0.00E+00	
Ag-110m	1.94E-04	2.50E+02	2.77E-03	6.00E-06	1.66E-08	2.77E-03
Te-127m	3.68E-04	1.09E+02	6.36E-03	9.00E-06	1.73E-13	1.92E-08
Te-129m	1.26E-03	3.36E+01	2.06E-02	7.00E-06	6.90E-34	9.85E-29
Te-129	1.84E-03	4.83E-02	1.44E+01	4.00E-04	0.00E+00	0.00E+00
Te-131m	3.25E-03	1.25E+00	5.55E-01	8.00E-06	0.00E+00	0.00E+00
Te-131	2.08E-03	1.74E-02	3.98E+01	8.00E-05	0.00E+00	0.00E+00
Te-132	3.83E-02	3.26E+00	2.13E-01	9.00E-06	0.00E+00	0.00E+00
Te-134	5.33E-03	2.90E-02	2.39E+01	3.00E-04	0.00E+00	0.00E+00
I-129	7.27E-09	5.73E+09	1.21E-10	2.00E-07	7.27E-09	3.63E-02
I-130	5.33E-03	5.15E-01	1.35E+00	2.00E-05	0.00E+00	0.00E+00
I-131	3.44E-01	8.04E+00	8.62E-02	1.00E-06	1.17E-127	1.17E-121
I-132	4.56E-01	9.58E-02	7.24E+00	1.00E-04	0.00E+00	0.00E+00
I-133	6.30E-01	8.67E-01	7.99E-01	7.00E-06	0.00E+00	0.00E+00
I-134	1.07E-01	3.65E-02	1.90E+01	4.00E-04	0.00E+00	0.00E+00
I-135	3.78E-01	2.75E-01	2.52E+00	3.00E-05	0.00E+00	0.00E+00

Radionuclide	Effluent Holdup Tank Concentration[1] (µCi/cm³)	Half-life[2] (days)	Decay Rate[3] (days⁻¹)	MPC[4] (µCi/cm³)	Groundwater Concentration[5] (µCi/cm³)	Groundwater Concentration/ MPC
Cs-134	3.35E-01	7.53E+02	9.21E-04	9.00E-07	1.50E-02	1.66E+04
Cs-136	4.85E-01	1.31E+01	5.29E-02	6.00E-06	1.17E-78	1.95E-73
Cs-137	2.42E-01	1.10E+04	6.30E-05	1.00E-06	1.96E-01	1.96E+05
Cs-138	1.79E-01	2.24E-02	3.09E+01	4.00E-04	0.00E+00	0.00E+00
Ba-137m	2.28E-01	1.81E-03	3.84E+02	NA[6]	0.00E+00	
Ba-140	4.85E-04	1.27E+01	5.46E-02	8.00E-06	4.20E-84	5.25E-79
La-140	1.50E-04	1.68E+00	4.13E-01	9.00E-06	0.00E+00	0.00E+00
Ce-141	7.76E-05	3.25E+01	2.13E-02	3.00E-05	4.02E-36	1.34E-31
Ce-143	6.79E-05	1.38E+00	5.02E-01	2.00E-05	0.00E+00	0.00E+00
Pr-143	7.27E-05	1.36E+01	5.10E-02	2.00E-05	1.25E-79	6.26E-75
Ce-144	5.82E-05	2.84E+02	2.44E-03	3.00E-06	1.53E-08	5.10E-03
Pr-144	5.82E-05	1.20E-02	5.78E+01	6.00E-04	0.00E+00	0.00E+00

Notes:
[1] Values from SSAR Table 2.4.13-1.
[2] Values from NUREG/CR-5512, Table E.1 (Kennedy and Strenge 1992); U. S. Department of Health Radiological Health Handbook (USDOH 1970) for Sr-92, Rh-106, and Ba-137m.
[3] Values calculated from SSAR Equation 2.4.13-4.
[4] Values from 10 CFR Part 20, Appendix B, Table 2, Column 2.
[5] Values calculated from SSAR Equation 2.4.13-5 for travel time of 9.25 years.
[6] Maximum Permissible Concentration (MPC) is not available.

Item #3 Table 2 Results of Transport Analysis Considering Radioactive Decay and Adsorption

Radionuclide	Effluent Holdup Tank Concentration[1] (µCi/cm³)	Backfill			Utley Limestone			Total Travel Time[5] (yr)	Ground Water Conc[6] (µCi/cm³)	Ground Water Conc / MPC
		Distribution Coefficient[2] (cm³/g)	Retardation Factor[3]	Travel Time[4] (yr)	Distribution Coefficient[2] (cm³/g)	Retardation Factor[3]	Travel Time[4] (yr)			
H-3	1.01E+00	0.0	1.0	9.16	0.0	1.0	0.09	9.25	6.01E-01	6.01E+02
Mn-54	6.77E-04	0.0	1.0	9.16	0.0	1.0	0.09	9.25	3.82E-07	1.27E-02
Fe-55	5.05E-04	0.0	1.0	9.16	0.0	1.0	0.09	9.25	4.70E-05	4.70E-01
Co-60	2.22E-04	1.4	7.6	69.48	3.9	63.4	5.75	75.24	1.15E-08	3.83E-03
Sr-90	2.38E-05	6.0	29.2	267.70	14.4	231.4	21.00	288.71	2.41E-08	4.82E-02
I-129	7.27E-09	0.0	1.0	9.16	0.0	1.0	0.09	9.25	7.27E-09	3.63E-02
Cs-134	3.35E-01	3.5	17.5	159.98	22.7	364.2	33.06	193.03	2.18E-29	2.42E-23
Cs-137	2.42E-01	3.5	17.5	159.98	22.7	364.2	33.06	193.03	2.85E-03	2.85E+03

Notes:
1 Values from SSAR Table 2.4.13-1.
2 Values from SSAR Table 2.4.13-3.
3 Retardation factors calculated using Equation 6 of Javandel et al. (1984) using a bulk density of 1.60 g/cm³ and effective porosities of 0.34 and 0.10 for the backfill and Utley limestone, respectively.
4 Travel times calculated as the product of the retardation factor and groundwater travel time (9.16 years for backfill and 0.09 years for Utley limestone).
5 Total travel times calculated as the sum of backfill and Utley limestone travel times.
6 Groundwater concentrations calculated using SSAR Equation 2.4.13-5 and total travel times.

Item #3 Table 3 Results of Transport Analysis Considering Radioactive Decay, Adsorption, and Dilution

Radionuclide	Groundwater Concentration[1] (µCi/cm³)	Surface Water Concentration[2] (µCi/cm³)	Surface Water Concentration / MPC
H-3	6.01E-01	5.45E-05	5.45E-02
Mn-54	3.82E-07	3.46E-11	1.15E-06
Fe-55	4.70E-05	4.26E-09	4.26E-05
Sr-90	2.41E-08	2.18E-12	4.37E-06
I-129	7.27E-09	6.59E-13	3.29E-06
Cs-137	2.85E-03	2.58E-07	2.58E-01

Notes:
[1] Values from Item #3 Table 2.
[2] Surface water concentrations calculated as the product of the groundwater concentration and the dilution factor (9.1E-05).

Item #3 Table 4 Compliance with 10 CFR Part 20

Radionuclide	Concentration / MPC	Radionuclide	Concentration / MPC
H-3	5.45E-02	Rh-106	0.00E+00
Cr-51	5.14E-37	Ag-110m	2.77E-03
Mn-54	1.15E-06	Te-127m	1.92E-08
Mn-56	0.00E+00	Te-129m	9.85E-29
Fe-55	4.26E-05	Te-129	0.00E+00
Fe-59	1.85E-22	Te-131m	0.00E+00
Co-58	4.18E-13	Te-131	0.00E+00
Co-60	3.83E-03	Te-132	0.00E+00
Br-83	0.00E+00	Te-134	0.00E+00
Br-84	0.00E+00	I-129	3.29E-06
Br-85	0.00E+00	I-130	0.00E+00
Rb-88	0.00E+00	I-131	1.17E-121
Rb-89	0.00E+00	I-132	0.00E+00
Sr-89	4.89E-19	I-133	0.00E+00
Sr-90	4.37E-06	I-134	0.00E+00
Sr-91	0.00E+00	I-135	0.00E+00
Sr-92	0.00E+00	Cs-134	2.42E-23
Y-90	0.00E+00	Cs-136	1.95E-73
Y-91m	0.00E+00	Cs-137	2.58E-01
Y-91	3.53E-17	Cs-138	0.00E+00
Y-92	0.00E+00	Ba-137m	0.00E+00
Y-93	0.00E+00	Ba-140	5.25E-79
Nb-95	3.36E-29	La-140	0.00E+00
Zr-95	5.03E-16	Ce-141	1.34E-31
Mo-99	0.00E+00	Ce-143	0.00E+00
Tc-99m	0.00E+00	Pr-143	6.26E-75
Ru-103	3.04E-26	Ce-144	5.10E-03
Rh-103m	0.00E+00	Pr-144	0.00E+00
		Sum of Ratios =	0.32

Notes:

1 Ratios for H-3, Mn-54, Fe-55, Sr-90, I-129, and Cs-137 are from Item #3 Table 3 and consider radioactive decay, adsorption, and dilution in Mallard Pond. Ratios for Co-60 and Cs-134 are from Item #3 Table 2 and consider radioactive decay and retardation. Ratios for the remaining radionuclides are from Item #3 Table 1 and consider radioactive decay only.

2 No MPCs are published for Rh-106 and Ba-137m. However, the half-lives for these radionuclides are short (less than one day) and they decay to near zero values. Their ratios have been taken as zero.

Item #4 Review the maximum hydraulic conductivity value used in the travel time analysis in SSAR Section 2.4.13 and provide a discussion as to why this value (2.65 ft/day determined from the ESP subsurface investigation program) was used as opposed to the maximum hydraulic conductivity value for the Utley Limestone (340 ft/day provided in the VEGP Units 1 and 2 UFSAR). Is the hydraulic conductivity value of 2.65 ft/day conservative?

Response:

As described in SSAR Section 2.4.13 of the ESP application, radionuclide travel times in the water table aquifer were estimated using the most conservative hydraulic conductivity value characterized for the ESP site. The hydraulic conductivity values for the ESP site were determined from in situ hydraulic conductivity (slug) tests performed in the groundwater observation wells installed at the site. The results of these tests are presented in SSAR Table 2.4.12-3 and summarized in Item #4 Table 1 of this response. Table 1 shows that the majority of the wells are screened in the Utley limestone, as indicated by the presence of shells and shell hash or Coquina, with the hydraulic conductivity values ranging from 0.12 to 2.65 ft/day. The maximum hydraulic conductivity of 2.65 ft/day was used in the travel time analysis.

Hydraulic conductivity tests performed in the Utley limestone for VEGP Units 1 and 2 are described in Section 2.4.12.2.4.3 of the UFSAR. The testing consisted of two pumping tests, and seven falling head and four constant head tests. The results are presented in UFSAR Table 2.4.12-13 and are summarized in Item #4 Table 2 of this response. The results of one pumping test indicate the possibility of localized, highly permeable zones in the Utley limestone based on hydraulic conductivity values ranging from 14,100 ft/yr (39 ft/day) to 125,400 ft/yr (343 ft/day). A second pumping test was performed in a less permeable zone of the limestone resulting in an estimated hydraulic conductivity of 3,250 ft/yr (9 ft/day). The falling head and constant head tests yielded hydraulic conductivity values ranging from 96 ft/yr (0.3 ft/day) to 5,800 ft/yr (16 ft/day).

Based on the hydraulic conductivities determined for the ESP site and VEGP Units 1 and 2, the hydraulic conductivity of the Utley limestone is highly variable across the site. However, to conservatively account for the potential presence of localized, highly permeable zones in the limestone with respect to the travel time analysis, the maximum value reported in the UFSAR, 125,400 ft/yr (343 ft/day) was used to re-estimate radionuclide travel times in the Water Table aquifer (see response to Item #3).

Response References:

None.

Item #4 Table 1 Hydraulic Conductivity Values for the Water Table Aquifer for ESP Site.

Observation Well No.	Test Interval [1] (ft bgs)	Material [2]	Hydraulic Conductivity [3] (ft/day)
OW-1003	72 - 91	"Reddish brown silty SAND (SM) with "Light tan silty SAND" with "Tan and grey clayey COQUINA"	0.12
OW-1005	143 - 169	"Pale yellow, silty SAND, calcareous (SM), fine-coarse-grained with shell pieces"	0.32
OW-1006	113 - 136	"Very light tan silty SAND (SM)" with "light gray COQUINA, unconsolidated" (OW-1006A) "Tan sandy and shelly CLAY (CH), saturated" with "Light tan, fine-coarse grained SAND with shell (SW)" (OW-1006)	1.40
OW-1007	99 - 120	"Tan fine-grained silty SAND (SM), saturated" with "Very light tan silty SAND (SM) becoming shelly" with "light olive grey CLAY (CH)"	2.65
OW-1009	81 - 98	"Very light tan silty SAND (SM)" with "Tan limestone shell hash, very light tan silty SAND (SM)" WITH "Brown silty CLAY".	1.10
OW-1010	70 - 92	"Tan poorly graded SAND with silt (SP-SM)" with "Brownish yellow clayey silty SAND (SC-SM), soft" with "White SHELL HASH"	0.18
OW-1012	71 - 94	"Brown SAND, fine-to-medium-grained with pale yellow silt (SM)" with "Pale olive silt (ML)" with "Pale yellow SILT, micaceous (ML)".	0.39
OW-1013	81 - 104	"Tan fine-to-medium-grained SAND (SP-SM) with tan or clay tubes or bioturbation" with "Light olive tan calcareous silty fine-grained SAND (SP-SM)" with "light olive tan calcareous CLAY (CL), wet but not saturated"	0.38
OW-1015	90 - 120	"Grayish white, fine-to-medium-grained SAND (SP) saturated" with Very light tan poorly graded SAND with silt (SP-SM)" with "Tan shelly (coarse) fine to medium grained clayey SAND (SC)".	0.44

Notes:
[1] Values from SSAR Table 2.4.12-3. (A typographic error was noted for OW-1006 and was corrected in this table).
[2] Material descriptions from the boring logs contained in SSAR Appendix 2.4A (report Appendix E).
[3] Values from SSAR Appendix 2.5A (report Appendix D).

Item #4 Table 2 Hydraulic Conductivity Values for the Utley Limestone for VEGP Units 1 and 2

Well No.	Test Interval (ft bgs)	Hydraulic Conductivity[1] (ft/yr)	Hydraulic Conductivity[1] (ft/day)
		Pumping Test Results	
1A	56 - 78	14,100	39
1B	68 - 78	125,400	343
1C	56 - 80	20,000	55
1D	56 - 80	44,100	121
2A	62 - 85	3,250	8.9
		Falling Head Test Results	
W-1	65 - 80	5,800	16
1A	63 - 78	600	1.6
W-2	69 - 85	980	2.7
2A	70 - 85	96	0.26
2B	69 - 84	360	1.0
2C	65 - 85	140	0.38
2D	70 - 85	2,100	5.7
		Constant Head Test Results	
1A	56 - 78	160	0.44
2A	56 - 85	3,200	8.8
2B	56 - 84	1,790	4.9
2D	56 - 85	1,190	3.3

Note:
[1] Values from UFSAR Table 2.4.12-13