CALCULATION PACKAGE:

Dike Failure Flood Modeling at the North Anna Service Water Reservoir

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Original draft on 4/4/2014, 1st revision on 3/4/2015

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1. Introduction

The Service Water Reservoir (SWR, or the reservoir hereafter) which is a safety-related facility is the normal source of water for the service water system. Therefore, adequate service water would be immediately available to maintain Units 1 and 2 in a safe condition even if Lake Anna were to be drained. The reservoir is partially enclosed by an earthen embankment dike. The dike is a Seismic Class I structure, and has been evaluated to preclude dike failure. Further conservatism for flood protection of the station is provided by an emergency dike and intercepting channel on the north slope of the reservoir.

The Flood Hazard Reevaluation Report (FHRR, 2013)[1] for the North Anna Power Station Units 1 and 2 states that the emergency dike and intercepting channel system has enough capacity to divert any potential reservoir dike breach outflow to the Waste Heat Treatment Facility Canal without adverse impact to the nuclear power plants. NRC staff performed in this report a hierarchical hydrologic modeling analysis to confirm that the emergency dike and intercepting channel system has sufficient hydraulic capacity to divert potential breach flood from the Service Water Reservoir. This document is to summarize the assumptions, data, methods, and results of the staff's dam failure flood analyses. This document is aimed to supplement the Staff Assessment report in relation to reviewing the FHRR. The staff's analyses here include breach parameter estimation, uniform flow calculation, HEC-RAS flood routing on the intercepting channel, and the impacts of breach flood on the plant site.

2. Description of the Service Water Reservoir

The FHRR as well as the response of RAI 3.4-1 (December 13, 2013: ML13357A100) and the latest version of UFSAR describe the concept and design of the SWR and the emergency dike and intercepting channel system. Figure 1 shows the layout of the North Anna Power Plant Units 1 and 2 with the reservoir and dike system. The reservoir was built to supply service water to the Units 1 and 2 during their normal operation, shutdown, and accident conditions. It is located approximately 500 ft south from the Units 1 and 2 power block area. The reservoir has an area of 7.9 acres, and is encompassed by 3000 ft long earthen embankment dike. The bottom elevation of the reservoir is approximately 305 ft above mean sea level (msl). The top elevation of the dike is 320 ft above the msl, with bottom elevations of the dike vary from place to plane, but the nominal bottom elevation of the dike in the postulated dike breach section is 305 ft msl. The normal maximum operation level of the reservoir is 315 ft msl.

The emergency dike and intercepting channel run from west to east on the steep slope area between the reservoir and the Units 1 and 2 plants. The intercepting channel is walled with about 10 ft high emergency dike on the left (north) and excavated and compacted natural slope on the right. The channel has a steep bottom slope from west to east. The licensee stated in the FHRR that the emergency dike and intercepting channel system was designed and constructed to divert any breach flood from potential failures of the reservoir dike in order to protect the Units 1 and 2 facilities.

The uppermost (west) part of the intercepting channel is connected to the SWR dike to prevent breach outflow from flowing to the plant site. The intercepting channel slopes downhill to the roadway east. Beyond the roadway, the channel drops steeply into the Waste Heat Treatment

[1] Virginia Electric and Power Company, 2013, Flood Hazard Reevaluation Report, Letter from Eugene S. Grecheck to U.S. Nuclear Regulatory Commission, March 11, 2013, ADAMS Accession No. ML13074A925.

Facility Canal. The intercepting channel has a bottom width of 40 feet and a total length of approximately 800 feet.

The area between the emergency dike and the plant site lays on a steep slope to the north, followed by an onsite drain area (named as the southeast (SE) subbasin in the FHRR) which is graded to drain any flood water in the area to the east into the Waste Heat Treatment Facility Canal. During the reevaluation, the licensee analyzed the capacity of this onsite drainage system using the HEC-RAS modeling. The north side of the SE subbasin is packed and lined to the west-east direction with many non-safety buildings, including Fuel Oil Storage Tanks, and Waste Disposal Building. These buildings, even though there are some gaps between them, could also be played as a barrier to interrupt breach outflows from entering into the power block area. Therefore, the licensee stated in the RAI response that the flood wave created by a potential reservoir dike failure will be diverted effectively by multiple barriers/walls, including intercepting channel, emergency dike, drainage area, and many onsite buildings. The main objective of this hydrologic analysis is to confirm the effectiveness of these diversion systems from potential SWR dike breach floods.

3. Postulating Dike Failure Scenarios

This section discusses plausible combined failure scenarios of the reservoir embankment dike and the emergency dike. To begin with, the staff considered three different dike failure modes applied to the SWR embankment dike: hydrologic failure (overtopping), seismic failure, and sunny day failure (piping). The staff screened out failure modes that are not plausible to the SWR dike. The staff applied similar failure modes to the emergency dike, and then formulated combined dike failure scenarios for testing with the hydraulic model.

3.1. Postulating SWR Dike Breach Scenarios

Hydrologic Failure (Overtopping)

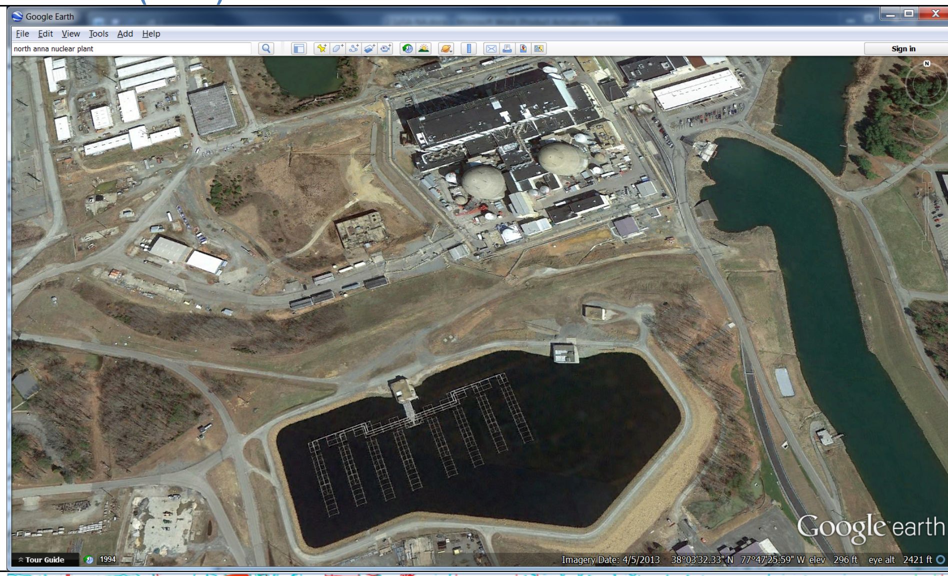
SWR is replenished by pumping from Lake Anna. SWR which is higher in elevation than the surrounding area has no basin inflow from the surrounding area other than direct precipitation into it. The licensee stated in the response of the RAI 3.4-1 that the SWR embankment dike was designed with a 5 feet freeboard above the normal maximum reservoir water level of 310 ft msl. The RAI response states that the freeboard was determined to cope with a probable maximum precipitation plus 40% antecedent PMP of 43 inches (3.58 ft). This design precipitation depth is smaller than the 5-feet freeboard. Therefore, the licensee concluded in the RAI response that the reservoir dike will not be overtopping.

In its review of the FHR and the RAI response, the staff noted that the above freeboard does not include the effects of wind wave. Therefore, the staff estimated the associated wind set-up height as following: First, the staff obtained the following two wind set-up parameters from the map provided in the FHRR:

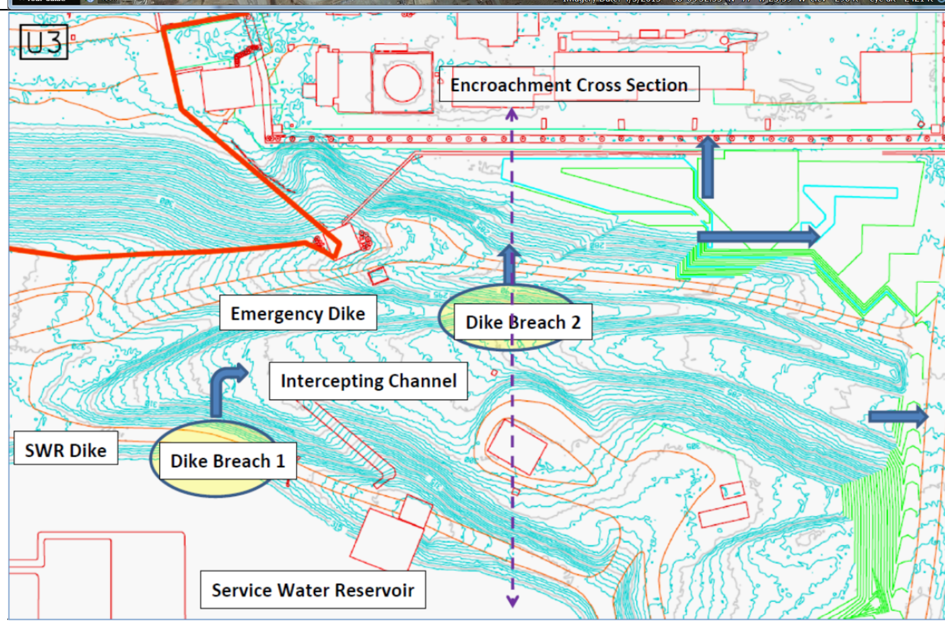
- Maximum fetch length of approximately 1000 ft obtained from the Google map (see Figure 1(a).
- Coincident 2-yr frequency wind speed of 50 miles per hour (mph) from the FHRR.

Figure 1. Site Features for North Anna Nuclear Power Station Units 1 and 2 with the Service Water Reservoir (SWR).

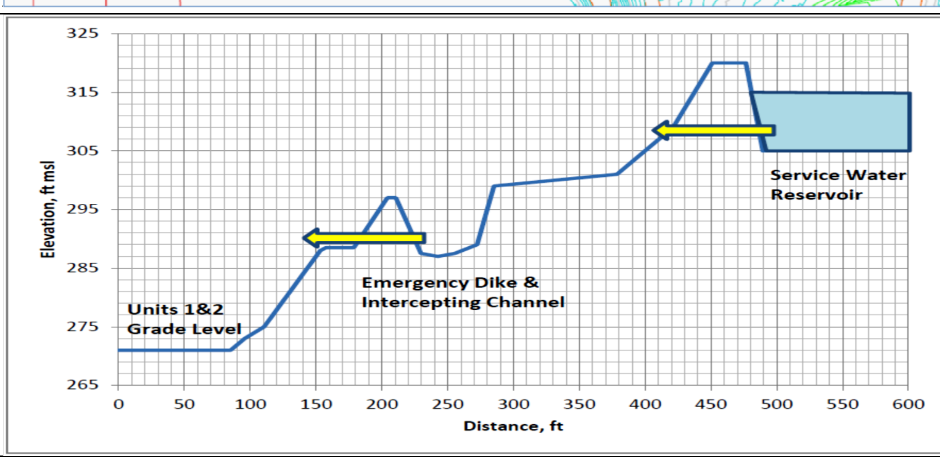
(a) Areal photographic map snapped from the Google Earth on 1/24/2014.



(b) SWR and the Emergency dike (Clipped from the FHRR Figure 2.1-2).



(c) Land surface profile at the encroachment cross section where a potential breach could occur.



The way of defining these two parameters follows the guidance provided in NUREG/CR-7046 (NRC, 2011[2]). With these wind parameters, the staff obtained a maximum wave run-up of 1.5 ft using the USACE Nomograph[3]. The sum of the PMP design depth and wind wave height is 5.08 ft. This new estimate exceeds slightly the 5-ft freeboard for the reservoir dike: However an erosion of the SWR dike will not occur because the depth of the overflow is shallow (e.g., 0.08 ft) and the wind set-up time is relative short (e.g., few minutes). Therefore the staff screened out a potential overtopping failure of the reservoir dike.

Seismic Failure

The licensee stated in the response to the RAI 3.4-1 that the SWR embankment dike is a Seismic Class 1 structure. That is, the reservoir and dike system was designed and constructed to preclude any structural failures from severe seismic activities. Further, the licensee is required by the regulation to instrument, inspect, and maintain the reservoir system consistent with the design requirements.

The licensee stated in the response to the RAI 3.4-1 that the site seismic reevaluation for the SWR system will be completed by March of 2013, and that the adequacy of the seismic qualification for the SWR embankment dike may have to be revisited based on the results of the seismic reevaluation and resulting station PRA reevaluations. Without having the result of the seismic reevaluation (at the time this study was done), the staff cannot screen out the seismic failure mode. Therefore, the staff performed hydrologic flood analyses with an assumption that the dike is failed by a design seismic event.

Sunny Day (Piping) Failure

The licensee stated in the response to the RAI 3.4-1 that the SWR embankment dike was designed as a Seismic Class I structure and is required to be instrumented, inspected, and maintained consistent with the design requirements. They also mentioned that the dike has been periodically inspected by site engineers and by external agencies responsible for inspection of such structures. The purpose of the inspection is to identify early indication of the presence of trees, rodent intrusion, or potential erosion areas on the dike to prevent weakening of the structure. Therefore, the licensee concluded in their RAI response that the design considerations and dike inspection and maintenance measures preclude conditions that may initiate piping failure.

However, the staff determined that the piping failure of the dike cannot be precluded completely during the lifetime of the plants. It is because piping failure in general is an episodic event that cannot be detected before it is fully developed. This staff view is consistent with the guide given in the JLD-ISG-2013-01 (page 6-1). The staff's determination is also based on the dike seepages observed from staff's site visits during the Early Site Permit review (undocumented). Therefore, the staff assumed that piping failure of the SWR dike is plausible. Correspondingly, the staff performed a hydrologic modeling analysis with a postulated dike piping failure as discussed in the next section.

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[2] USNRC (U.S. Nuclear Regulatory Commission), 2011e, "Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United State of America," NUREG/CR-7046, November 2011

[3] USACE, "Shore Protection Manual", U.S. Army Engineer Waterways Experiment Station, 3rd ed. U.S. Government Printing Office, Washington, DC, 1977.

Reservoir dike Failure Scenarios

As both seismic and piping failure modes of the SWR dike cannot be screened out, the staff postulated a conservative dike failure scenario for the forthcoming hydraulic modeling:

- The failure of reservoir dike is assumed to be initiated by either piping or seismic event.
- Dike breach would occur on the northern section of the dike between the pump house and the junction of the reservoir dike and the emergency dike (see Figure 1(b)). Breach on this critical section could potentially inundate the plant site, while flood occurring from the outside of this critical section will not impact on the plant site.
- Initial reservoir water level is at the normal maximum operation level of 315 ft msl.
- The reservoir dike is breached in a trapezoidal shape (i.e., wide top and narrow bottom with 45 degree side angle), extending to the assumed bottom of the dike (at the elevation of 305 ft msl on the postulated breach location). Staff assumed that breach is not extended to the foundation of the dike as the foundation soils was compacted with very low permeability (UFSAR: Dominion, 2011)[4].
- Breach parameters (e.g., breach width and time) are determined by an average of several empirical breach equations.

3.2. Postulating Emergency Dike Failure Scenarios

The emergency dike was built on a steep slope area. Unlike the reservoir dike, the emergency dike is not a Seismic Class I structure. The staff considered the failure of the emergency dike combined with the postulated SWR dike failure. As before, the staff considered the following three dike failure modes:

- Hydrologic failure (overtopping): During the design of the emergency dike and intercepting channel system, The licensee estimated, for the reservoir dike, a peak breach outflow rate of approximately 1000 cfs (UFSAR Unit 1&2, 2011). This estimate was based on an assumption that the reservoir dike would fail in a 90-degree v-notch shape. In its FHRR review, the staff noted that the breach outflow rate would be about 10 times larger than the licensee's estimate (see the next section). Because of this increased breach flood rate, the staff performed a hydrologic modeling to confirm that the capacity of the emergency dike and intercepting channel system is sufficient to prevent any overtopping through the emergency dike.

- Seismic Failure: The staff determined that a combined failure of both reservoir dike and emergency dike by a seismic failure is plausible.

- Sunny-day Failure (Piping): The staff noted that sunny day failure of the emergency dike itself during the normal (no-flooding) condition is not plausible because the emergency dike is in dry condition. Simultaneous sunny day failures of both reservoir dike and

[4] Dominion, 2011, "North Anna Power Station Units 1 & 2, Updated Final Safety Analysis Report," Revision 47, September 27, 2011, ADAMS Accession No. ML112870511. (Not Publicly Available)

emergency dike are also not plausible because they are independent. Therefore sunny day failure of the emergency dike was excluded for the forthcoming hydraulic analyses. Even with the SWR dike failure (either by piping or seismic event), the emergency dike will not failed by piping because the inundation time of the emergency dike from a upstream flooding is too short (~20 minutes) to incur any piping failure of the emergency dike. The staff noted that piping failure of the emergency dike would occur if there is a prior crack on the dike by an earthquake event. Thus, the staff performed a hydrologic analysis to identify the effects of the emergency dike failure created by a crack and piping event.

3.3. Postulate Combined Failure

Based on the discussions above, the following combined dike failure scenarios were postulated for the hydraulic model simulations:

1) Sunny day failure of reservoir dike could trigger an overtopping of emergency dike.
2) Seismic failure of reservoir dike could trigger an overtopping of emergency dike.
3) Because the emergency dike is not a Seismic Class I structure, it is plausible that emergency dike would fail if reservoir dike is failed by a seismic event. A seismic event could create either cracking or rupture/sliding emergency dike. The gap created by the seismic event will be further widened by breach outflows from reservoir dike. Without knowing the size of rupture/sliding, the staff postulated three different gaps as discussed later.

The postulated scenarios are drawn in Figure 2. The staff performed the subsequent hydrologic flood hazard analyses based on these failure scenarios.

Figure 2. Plausible combinations of dike failure scenarios considered in the hydrologic analyses, where solid line is highly likely event while dot lines are less likely events.

Failure Mode	SWR Dike	Emergency Dike
Hydrologi Failure (Overtopping)	NO	YES
Seismic Failure	YES	Rupture/Sliding: NO Crack-Piping: YES
Sunny Day Failure (Piping)	YES	NO

4. Estimating Dike Breach Parameters

In general, dam (or levee) breach parameters to predict downstream flood hazards include width, slope, depth, time, and peak outflow of breach. These parameters can be estimated by empirical regression equations, semi-analytical models, or full analytical models, or their combinations. The NRC staff estimated SWR dike breach parameters using empirical equations listed on the dam breach analysis guide by USBR (1998[5]). These equations were derived using historical dam breach data. The independent variables of the regression equations are reservoir size values, including storage volume, head, breach type, embankment materials. The dam breach regression equations are by and large uncertain but simple and useful enough to determine breach parameters conservatively.

Table 1 summarizes dimensions of the SWR and its dike system presented in the FHRR and RAI response. Tables 2 lists breach equations with estimated parameter values. Table 3 gives breach peak outflow estimates. Taking an average of estimations using different equations, the staff determined the following SWR dike breach parameters:

- Breach width: 60 ft
- Breach time: 0.24 hours (or 14 minutes)
- Peak breach outflow rate: 10,319 cfs

Section 3.8.4.7.5 of the UFSAR (Dominion, 2011)[6] states that the emergency dike and intercepting channel system was designed to pass a flood rate of approximately 1000 cfs under the assumption that the SWR dike would breach in a 90-degree v-notch shape. However, the staff conservatively assumed a rectangular breach shape with the above breach parameter. This breach scenario results in approximately 10 times larger breach peak outflow than the licensee's estimate, sufficiently large enough to justify a detailed hydraulic analysis to confirm the diversion capacity of the emergency dike.

Table 1. Dimension of the Service Water Reservoir and its dike.

Parameter	Value in English Unit	Value in Metric Unit
Normal Maximum Operation Level	315 ft msl	96 m msl
Bottom Elevation	305 ft msl	93 m msl
Top Embankment Elevation	320 ft msl	97.5 m msl
Dike Cross-section Area	1,400 ft^2	130 m^2
Breach Head (h_w or h_b)	10.0 ft	3.05 m
Breach Volume (v_w)	108,504 ft^3	10,080 m^3

Note: msl stands for mean sea level.

[5] USBR (U.S. Bureau of Reclamation), 1998, Prediction of Embankment Dam Breach Parameters: A Literature Review and Needs Assessment", DSO-98-004, Dam Safety Research Report, U.S. Department of the Interior, Bureau of Reclamation, Dam Safety Office.

[6] Dominion (Virginia Electric and Power Company), 2011, "North Anna Power Station Units 1 & 2, Updated Final Safety Analysis Report," Revision 47, September 27, 2011, ADAMS Accession No. ML112870511. (Not Publicly Available)

Table 2. Estimation of SWR dike breach parameters.

Equation	Breach Width (ft)	Breach Time (hr)
Johnson & Illies (1975)	30	n.a.
Singh & Snorrason (1984)	n.a.	0.25
MLM (1984)	12	0.17
FERC (1987)	50	n.a.
USBR (1988)	30	0.10
Froehlich (1995)	30	0.35
von Thun & Gillette (1990)	205	0.31
Average	60	0.24
Average w/o MLM or vT&G	35	

Table 3. Estimation of SWR dike breach outflow rates (Qp).

Method	Equation	Qp (cms)	Qp(cfs)
Kirkpatrick(1977)	$Qp=1.268\ (h_w+.3)^{2.5}$	26	913
SCS(1981)	$Qp=16.6\ (h_w)^{1.85}$	130	4,585
Hagen(1982)	$Qp=.54\ (S\ h_d)^{.5}$	310	10,954
USBR(1982)-Env.	$Qp=19.1\ h_w^{1.85}$	149	5,276
Singh & Snorrason (1984)	$Qp=13.4\ h_d^{1.89}$	110	3,870
Singh & Snorrason (1984)	$Qp=1.776\ S^{.47}$	413	14,592
MLM (1984)	$Qp=1.154\ (v_w\ h_w)^{.412}$	217	7,652
Costa (1985)	$Qp=1.122\ s^{.57}$	832	29,392
Costa (1985)	$Qp=.981\ (s\ h_d)^{.42}$	204	7,201
Costa (1985)	$Qp=2.634(s\ h_d)^{.44}$	706	24,928
Evans (1986)	$Qp=0.72\ v_w^{.53}$	336	11,862
Froehlich (1997)	$Qp=.607\ v_w^{.295} h_w^{1.24}$	74	2,603
Average		292	10,319

Note: $h_w=h_d$, and $v_w=S$.

5. Steady Uniform Flow Analysis on the Intercepting Channel

The staff used a hieratical modeling approach to confirm the capacity of the intercepting channel. The approach starts with evaluating the channel conveyance capacity using a simple analytical flow equation. The approach is further refined using a numerical hydrodynamic model with realistic breach scenarios and geometric data. For the hydrologic modeling, the staff considered both steady and unsteady flow conditions on the intercepting channel. This section discusses the first step of the hierarchical approach.

FHRR states that the bottom width of the intercepting channel is about 40 ft. The width increases with increasing elevation due to the slopes on both sides. The actual top widths channel vary along the channel reach depending on the topographic features as shown on

Figure 1(b). That is, the channel width is about 150 ft at the upstream (just below the SWR reservoir), then reduces to about 40 ft at the mid-reach and thereafter. The staff determined this encroachment section as a potential overtopping spot on the intercepting channel because this section will create contraction of flows with faster velocity and higher hydraulic head. Therefore, the staff computed flow rate and water level at this encroachment section. The staff first used a Manning's equation[7] which is widely applied to a uniform, mile-slope flow condition.

The staff estimated the following hydraulic parameters using the cross-section dimensions on the encroachment section from a detailed topographic map provided by the licensee (an electronic version of the FHRR Figure 2.1-2):

- Dike top elevation: 297 ft msl
- Bottom slope (S): 0.0578
- Cross section area (A): 526 ft^2
- Perimeter (P): 126.7 ft
- Hydraulic radius (R): 4.155

The bottom of the intercepting channel is bare soils that are covered with lawn or small vegetation. The bottom roughness condition of this type of channel fits the roughness category C-a (excavated, earth, straight channel) on the Table 5-6 of Chow (1959)[8]. Therefore staff selected for a sensitivity analysis n-values ranging from 0.01 to 0.04. It should be noted that Manning's n value of 0.01 is unrealistically low for an unimproved, excavated channel but selected here for conducting an analysis of discharge to Manning's n. Using these n-values and the above hydraulic parameters, the staff estimated flow rates as summarized below.

Table 4. Intercepting channel flow rates estimated with different n-values.

n-value	Flow Velocity, V, ft/s	Flow Rate, Q=AV, ft^3/s	Froude Number[9], $Fr=V/(gD)^{0.5}$
0.010	92.3	48,591	5.1
0.020	46.2	24,295	2.6
0.030	30.8	16,197	1.7
0.035	26.4	13,883	1.5
0.040	23.1	12,148	1.3

The result of this analysis indicates that the channel conveyance capacities for the range of plausible n-values exceed the estimated peak breach outflow of 10,319 cfs. However the channel flows could be a supercritical condition with unrealistically high flow velocities. This is mainly due to a steep channel slope. Therefore, the staff determined to use a 1-D numerical modeling approach that can incorporate the supercritical flow condition as discussed in the following section. The 1-D hydraulic model was chosen for simplicity but the staff believe that this model may be sufficiently accurate to simulate breach outflows conservatively.

[7] Manning equation: $V=1.486\ ^{R2/3}S^{1/2}/n$, where n is the Manning's roughness coefficient.

[8] Chow, V-T, 1959, Open Channel Hydraulics, McGraw-Hill Book Company, New York.

[9] The Froude number value is used to determine either subcritical (Fr<=1) or supercritical (Fr> 1) flow, where g=32.15 ft/s, and D is the flow depth in ft.

6. HEC-RAS Steady Simulation on the Intercepting Channel

The staff adopted the HEC-RAS model to simulate breach flows on the intercepting channel. HEC-RAS (USACE, 2012)[10] was developed to perform one-dimensional hydraulic calculations for a full network or natural and constructed channels. HEC-RAS can simulate both steady and unsteady flows on either subcritical or supercritical conditions, or mixture of the two conditions along different reaches. The model has various simulation options, including many hydrologic structure flows, sediment transport, and water temperature analysis. The staff used the HEC-RAS reservoir routing and dam breach options here.

The staff first performed a bounding steady HEC-RAS simulation to check the capacity of channel diversion. The staff used a steady peak breach outflow rate of 10,319 cfs as an upstream inflow boundary condition in the model. This peak flow in a steady condition bounds any unsteady breach outflow hydrographs. The staff selected this approach because steady simulation generally guarantees numerical stability.

Model Setup

For simplicity, the staff used an average of the peak breach outflow rates estimated using empirical regression equations as a steady upstream inflow boundary condition. The staff created 14 cross-sections to represent the entire intercepting channel (see Figure 3 top). The first downstream cross section (Reach Station 0 mile) is at the exit boundary on the access road near the Waste Heat Treatment Facility. The last upstream cross section (Reach Station 0.151 mile) is an upstream inflow boundary located in the SWR, representing an imaginary upstream boundary condition. The geometric channel cross sections (e.g., distances and elevation pairs) were obtained from the electronic version of the topographic map shown on the FHRR Figure 2.1-2. The staff selected n-value of 0.025 for the intercepting channel, but performed a sensitivity analysis with varying n-values.

Simulation Results

The staff simulated HEC-RAS for three flow conditions: subcritical, supercritical, and mixed conditions. The result of simulations is summarized on Table 5. This result indicates that the simulation with a subcritical option creates overflow at the encroachment section (the cross sections #6, #51, #5 as marked in yellow on the table) as in the case of the Manning's equation. The staff also observed high flow velocities and Froude numbers as in the Manning's equation case. Table 5 indicates that both supercritical and mixed condition runs are nearly identical to the subcritical case. However, the staff determined that a mixed flow option is the most appropriate for the intercepting channel flow because the flow on the reach are mixed modes as were shown on the model output. The staff found no overtopping on the emergency dike for both supercritical and mixed runs, indicating that the intercepting channel has a sufficient diversion capacity.

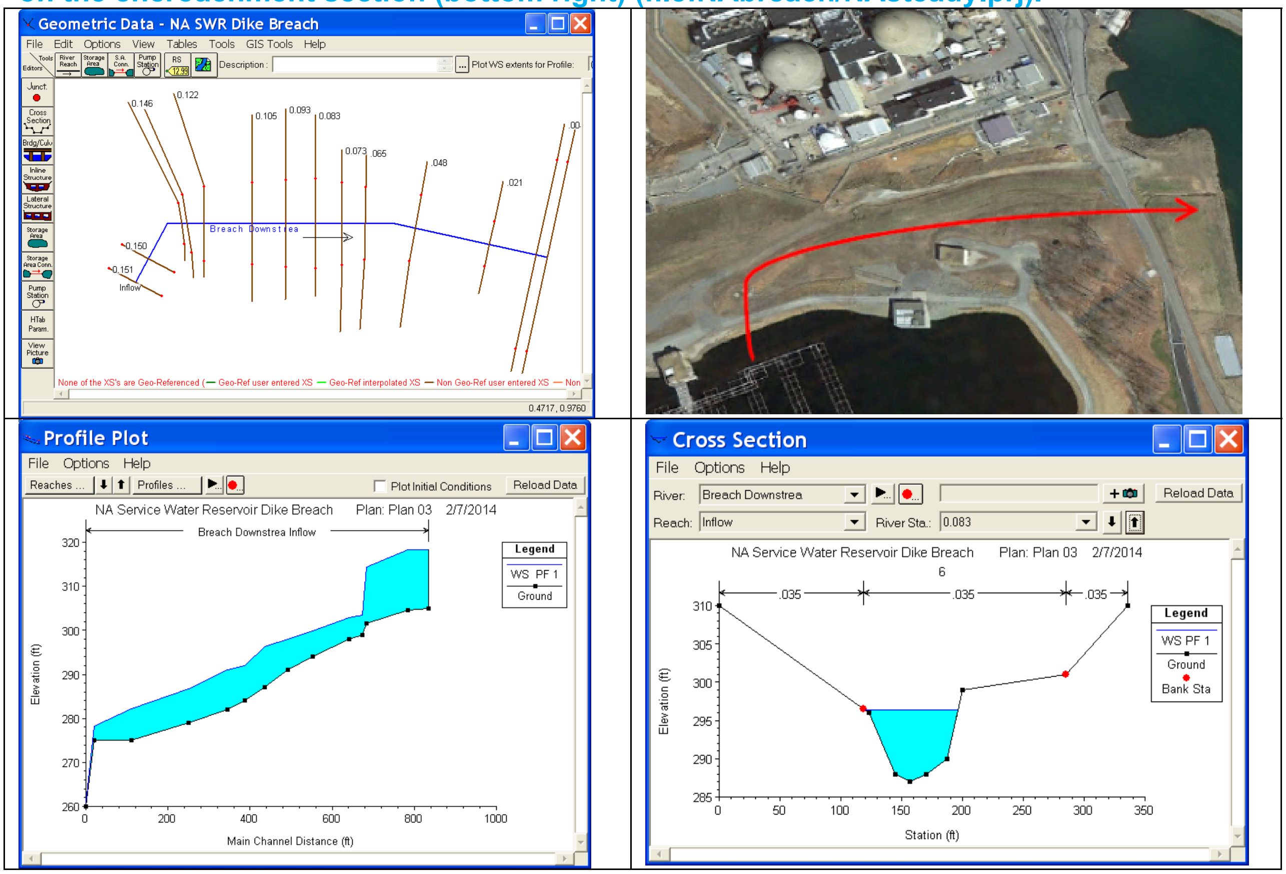
[10] USACE, 2010, HEC-RAS Version 4.1 User's Manual, Hydrologic Engineering Center, Davis, CA.

The plot on Figure 3c (bottom left) shows a water surface profile along the intercepting channel, where the water surface profile was simulated using the mixed subcritical-supercritical option. This figure also presents the longitudinal channel bottom cross section with simulated flood levels at the encroachment section as a display. It indicates that the simulated water level is below the top of the emergency dike (red dot on the left side of the channel). This result indicates that the breach outflow conservatively estimated using empirical equations will not overtop the emergency dike. Correspondingly, the staff concluded that the intercepting channel has a sufficient diversion capacity for a conservatively estimated peak breach outflow. In other words, the breach outflows created by seismic or piping failure of the SWR dike will not overtopping the emergency dike. This is true only when the emergency dike system is not failed simultaneously with the SWR dike failure. The emergency dike failure case is considered on the next section.

Table 5. Simulated water levels with different flow conditions.

Section ID #	Distance, mi	Channel Elev., ft	Dike Elev., ft	Simulated Water Surface Elevation, ft		
				Sub-critical	Super-Critical	Mixed
11	0.151	305	320	318.7	309.4	318.7
10	0.15	305	320	318.8	308.9	318.8
9	0.146	304	316	315.1	315.1	315.1
8	0.128	299	309	307.1	303.4	303.4
71	0.122	298	307	306.2	302.8	302.8
7	0.105	294	303	302.3	299.8	299.8
61	0.093	291	300	300.6	298.0	298.0
6	0.083	287	297	298.5	296.4	296.4
51	0.073	284	293	295.0	292.0	292.0
5	0.065	282	292	294.0	291.1	291.1
4	0.048	279	289	290.0	286.8	286.8
3	0.021	275	284	285.0	282.2	282.2
2	0.004	275	282	280.1	278.2	278.2
1	0.000	260	310	263.2	260.9	260.9

Figure 3. HEC-RAS steady simulation with a peak breach outflow: (a) map showing HEC-RAS channel (top right), (b) geometric configuration of the intercepting channel (top left), (c) simulated water surface profile (bottom left), and (d) cross section with simulated flow on the encroachment section (bottom right) (file:NAbreach/NAsteady.prj).



7. HEC-RAS Unsteady Simulation for a SWR Dike Failures

The staff used a HEC-RAS unsteady option to simulate SWR dike breach outflows and downstream flows on the upper part of the intercepting channel. The purpose of this simulation is to obtain a breach outflow hydrograph which are in turn used as an upstream inflow for the subsequent intercepting channel modeling and for the onsite inundation.

In these simulations, the staff assumed that both dikes fail by a seismic event: either abrupt large rupture or a crack-piping failure, or a combination of two as discussed in the Section 3.2. The source of water in the intercepting channel is from the SWR dike breach outflow.

The intercepting channel has a very steep slope (S=0.0578) with varying cross section widths along the reach. In addition, the SWR dike breach outflow rates are changed rapidly due to a fast breach growth (e.g., full breach time of about 0.24 hours). Numerical solution of a highly unsteady flow tends to be unstable or cannot even get a convergent solution. The staff initially tried to setup a HEC-RAS model that covers the entire channel reach with two breaches on the

SWR dike and emergency dike. However the staff was unable to get a convergent solution. One of the solutions in such an instability case is to use smaller reach intervals between cross sections or to set small simulation intervals or to relax acceptable simulation errors. This process tends to be highly subjective in simulation. Therefore, the staff adopted an incremental simulation approach where the staff used three independent HEC-RAS models: an upstream model with a SWR breach, a downstream model with an emergency dike breach, and a southeast (SE) subbasin model. Simulating the three models in series is possible because the upstream flow influences downstream flow but not vice versa. Detailed modeling process and simulation results are discussed in the following subsections.

7.1. Reservoir Dike Breach Outflow Simulation[11]

The purpose of this HEC-RAS model is to simulate a SWR dike breach outflow hydrograph under the postulated SWR dike failure caused by a seismic or piping event. The staff used a HEC-RAS unsteady option as the breach outflows change rapidly due to a fast breach growth rate. The model incorporates only the upper part of the intercepting channel, approximately 800 ft in length which is enough to simulate the breach outflow with the effects of the downstream backwater on the intercepting channel. This channel reach is represented by eight cross sections in the model.

The staff used the breach parameters (e.g., width and time of breach) estimated by empirical regression equations in the Section 4. The SWR dike was modeled as an internal weir structure which has a top elevation of 320 ft msl. The two uppermost cross sections represent the SWR and its dike. In the model, the effect of breach outflow in the SWR is represented by a storage-volume curve with a specified initial reservoir water level. The initial water level conditions of these two sections were set to the initial reservoir level of 315 ft msl. The following summarizes breach data specified in the HEC-RAS model:

- Final breach bottom width: 50 ft assuming 1:1 side slope on each dike side
- Final bottom breach elevation: 305 ft msl
- Breach weir coefficient: 2.6
- Full breach formation time: 0.24 hours
- Piping failure with piping coefficient of 0.5 (not sensitive)
- Initial piping elevation: 310 ft msl
- Failure trigger time at t=0 hour

It should be noted that the 60 ft breach width determined from the regression equations is an average of the trapezoidal breach section (later to the flow direction), with 45 degree side slope (e.g., 50 ft bottom wide and 70 ft top width). In addition, the staff set the reach downstream end as a “normal depth” boundary condition with a small frictional slope (e.g., S=0.0001) to mimic

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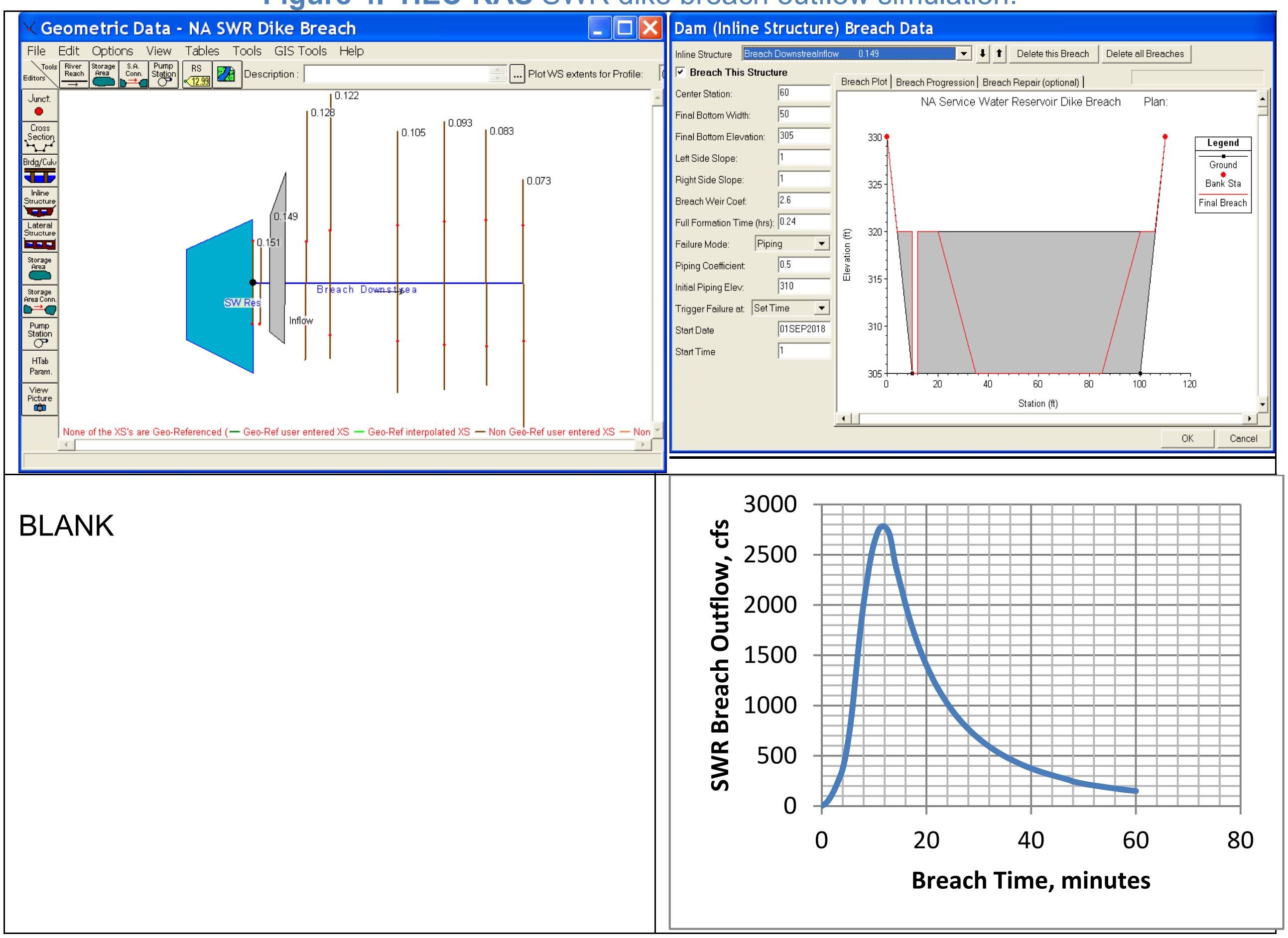
[11] File: C:\hec\HEC-RAS\NAbreach\NAunsteadyBREACH2.prj

the actual exit condition of the model (e.g., no backwater effects from delaying the exiting flow). The staff set zero initial flow on each reach, and specified an initial reservoir level of 315 ft msl at the beginning of the simulation. The model was set up so that a breach process starts at the beginning of the simulation. The model was simulated with the one second time intervals for 2 hours. The staff used a mixed flow regime option as the result of the previous steady simulation shows both subcritical and supercritical flow conditions. The staff used a sine breach growth curve which is more conservative than that of the linear curve in terms of simulated flood levels.

Figure 4 shows features and the result of the HEC-RAS simulation. The staff used a small gap (e.g., one ft width: Figure 4 top right) on the dike to prevent an unstable solution but not to notably increase breach outflows. Figure 4 (bottom) presents the simulated breach outflow hydrograph for a postulated SWR embankment dike failure. From this simulation, the staff obtained a peak breach outflow rate of 2,800 cfs and peak time of 12 minutes. It should be noted that the model simulated peak time (e.g., time to peak breach outflow) is smaller than the estimated average breach time (e.g., time from initiation to completion of a breach) because the water level in this small reservoir drops quickly with a good-sized (final 60 ft width) breach. The simulated outflow hydrograph was used as an upstream input to the subsequent models.

Figure 4. HEC-RAS SWR dike breach outflow simulation.



7.2. Emergency Dike Breach Simulation[12]

The purpose of this simulation is to obtain a breach outflow hydrograph through a postulated emergency dike breach gap. The emergency dike is assumed to be failed by a seismic event that creates a large rupture (a total demolition of the dike from the top to the bottom for a give width) or a small crack (a few feet or less) in the dike system. The small crack is assumed to be expanded by a piping failure process when the breach outflow from a postulated SWR dike failure immerses and inundates the emergency dike. As before, the staff determined that an emergency dike breach occurs at the encroachment section as this cross section is subject to have higher flow velocity and head due to the contraction effect, thus vulnerable.

Because the emergency dike is dry most of its lifetime and the ground water levels around the dike is below the embankment, it is highly unlikely that the emergency dike would be ruptured entirely to the level of dike bottom by a seismic event. However, this study conservatively postulated that the 10-ft-high emergency dike is totally ruptured to the bottom with a maximum breach width of up to 60 ft as was postulated for the SWR dike failure. Also the staff considered a scenario of seismically induced cracking of the emergency dike, that will develop a piping failure with a large gap (60 ft width) during the flooding. In sum, the staff postulated and analyzed the following four conservative breach scenarios:

- Scenario 1: Instantaneous rupture of the emergency dike with a 20 ft width gap
- Scenario 2: Instantaneous rupture of the emergency dike with a 40 ft width gap
- Scenario 3: Instantaneous rupture of the emergency dike with a 60 ft width gap
- Scenario 4: Seismically-induced crack on the dike, then expanded by the piping process

For the simulation of these scenarios, the staff modified the HEC-RAS channel geometric configuration established in the previous subsection. The modifications include (1) removing the SWR and its dike breach option, (2) using the SWR dike breach outflow hydrograph in Figure 4 bottom as an upstream inflow boundary condition, and (3) adding the emergency dike as a lateral weir and its failure scheme. This modified HEC-RAS setup gives better stability in numerical solution compared to the model that simulates two breaches (on both SWR dike and emergency dike) together.

To simulate an emergency dike breach, the staff used either an open gap on the weir for the Scenarios 1 through 3 or the HEC-RAS breach option for the Scenario 4. That is, for the first three scenarios, the lateral weir representing the emergency dike is pre-opened at the starting of the simulation in a rectangular shape with a specified width (e.g., 20, 40, 60 ft) at the beginning of the simulation. The model was set up so that the channel flow at the dike breach section is divided into two parts: downstream channel flow and breach outflow. The dividing

[12] Files: C:\hec\HEC-RAS\NAbreach\NAunsteadyBREACH31.prj
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flow is governed by the conveyance capacity of these two downstream sections (i.e., cross section area, slope, and roughness of two pathways).

Setting up an emergency dike breach for the Scenario 4 is similar to that of the SWR dike breach, with the following specifications:

- (Breach initiation time): Specifying a realistic breach initiation time is very critical in breach simulation. The staff used a specified trigger failure at the "set time" option in HEC-RAS. From the previous simulation, the staff determined that breach initiates at 8 minutes from the starting of the simulation, which is also equivalent to the time from the beginning of the SWR breach. This is the time needed to reach the initial SWR breach outflow to the encroachment section. The staff used a sine curve as a representative for the piping progression process. This curve gives more conservative peak breach outflow compared to the linear one.

- (Breach bottom elevation): The bottom of the emergency dike breach would be sloped following the channel bottom slope. However the HEC-RAS breach option allows only a horizontal bottom to the main channel direction between the two cross sections 9one upstream and one downstream to the breach spot), even though the dike represented by a lateral weir can be sloped in the model. The staff performed a sensitivity analysis using HEC-RAS with varying bottom elevations. The result revealed that the breach bottom slope is very sensitive in determining breach outflows (see Figure 6). This sensitivity result indicates that the bottom elevation of 286 ft gives unrealistically high breach outflow, while the elevation of 290 ft results in too small outflows. Therefore the staff selected the breach bottom elevation of 288 ft msl which deems an average elevation with the breach section along the dike.

- (Breach downstream boundary condition): The downstream area of the emergency dike breach section has a steep slope to the north, thus breach outflow is not affected by the downstream condition. Therefore the staff assumed an exit boundary condition with a specified energy slope of 0.0001 so that the breach outflow is not interrupted (free of interruption) by this exit boundary condition.

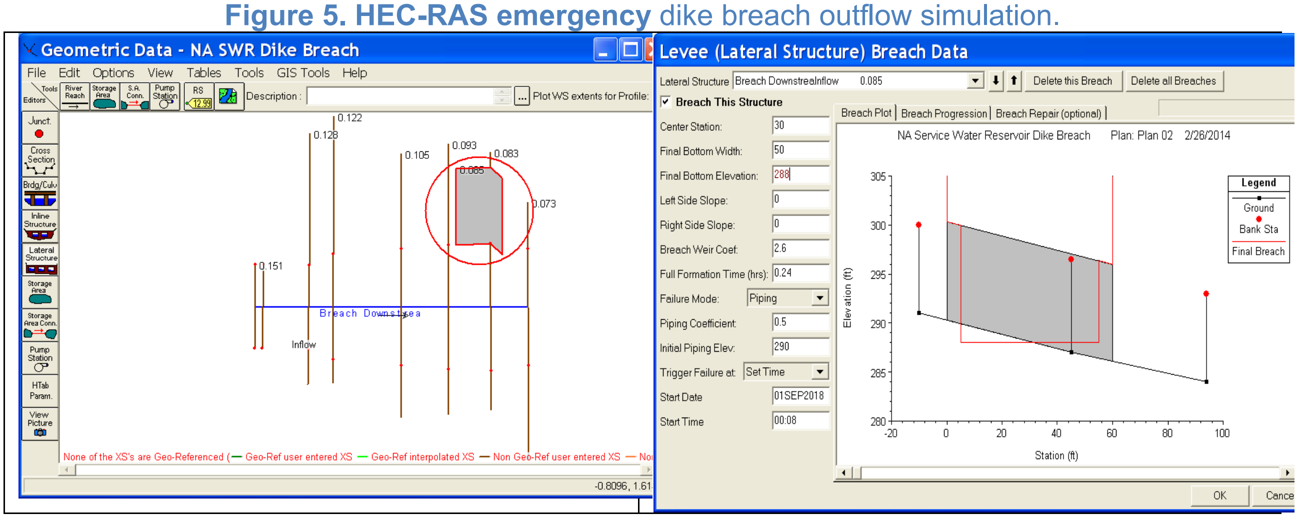
It should be noted that the HEC-RAS dam breach option allows a lateral structure, such as a levee, to have an upstream to downstream slope, but does not allow the bottom of a levee breach bottom to be similarly sloped the actual bottom of the levee. A series of narrow breaches running from the upstream station on the levee to the downstream station on the levee, each having a different base elevation, could capture more accurately the slope of the lateral structure. However, the current version of HEC-RAS does not accommodate multiple breaches on a single lateral structure. If the accuracy of the breach simulation is needed, one should adopt a detailed three dimensional model that simulates a coupling of the breach process and flow processes on a steep slope: However that kind of model is not readily available to the staff.

Figure 5 bottom shows hydrographs for upstream and downstream flows at the emergency dike breach section. It should be noted that the peak breach outflow is delayed from the inflow peak due to a gradual breach process. Table 6 summarizes fractions (in per cents) of breach outflow through a gap on the emergency dike by scenario. The breach outflows ranging from 9 per cent to 30 per cent of the total SWR breach outflow, with the best estimate of 21 per cent. The total breach outflow is for one hour simulation duration for the comparison purpose. However, it should be noted that there is a small amount of residual flows after the one hour simulation as shown on Figure 5 bottom.

Table 6. Simulated emergency dike breach outflow volumes with their factions to the total reservoir storage volume of 3,033,000 ft^3.

Scenario	Total Breach Outflow, ft^3	Fraction of breach outflow in per cent
1	280,000	9
2	551,000	18
3	897,000	30
4	649,000	21

Figure 5. HEC-RAS emergency dike breach outflow simulation.



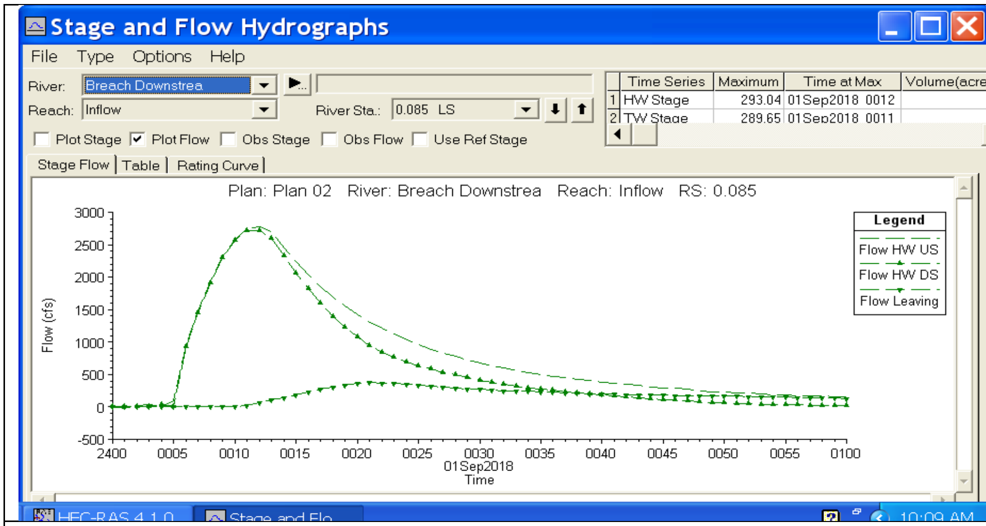
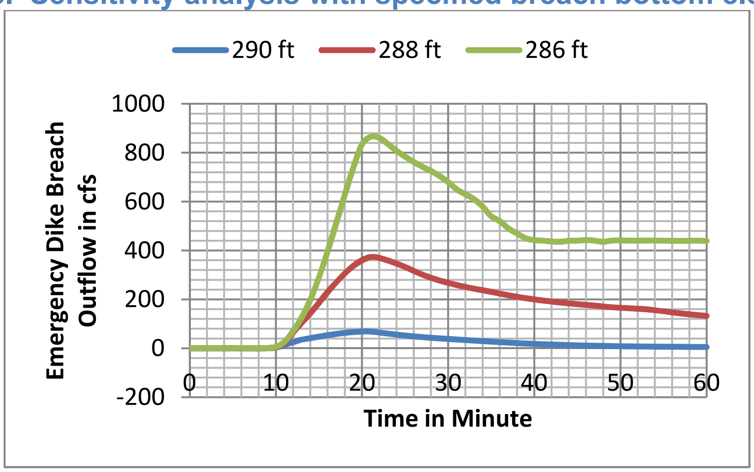


Figure 6. Sensitivity analysis with specified breach bottom elevations



8. Effects of Dike Failure Flood on the Plant Site

The staff analyzed the effects of breach outflow from the SWR on the plant site. In this analysis, the staff assumed both SWR dike and emergency dike fail simultaneously, and that the breach outflow reaches to the southeast corner of the plant site, namely SE1 and SE2 in Figure 7 (top). These subbasins receive potential breach outflow from the south, then drain water to the Waste Heat Treatment Facilities Canal. The HEC-RAS model was used to simulate the depth of temporary inundation at the SE1 SE2 subbasins. As before, the staff performed a bounding steady simulation first, and then refined the model to simulate detailed and realistic unsteady flows.

8.1. Steady Simulation[13]

The licensee defined in the FHRR a total of 22 subbasins within the Units 1 and 2 areas for the local intense precipitation flood analysis. Among these subbasins, four subbasins (namely SE1, SE2, SE3, SE4 (Figure 7 top two plots) are of interests here as these subbasins could receive dike breach outflow. The total area of these two basins is approximately 136,500 ft^2. The water from these subbasin areas could be discharged directly into the Waste Heat Treatment Facility Canal. The licensee named the drainage channels on these four subbasins as the SE Basin and its tributary (see Figure 7 middle).

The staff simply took the SE drainage portion of the licensee's HEC-RAS model, and then modified the model by setting peak breach outflow obtained from the previous simulation as an upstream inflow to the uppermost reach (Reach Section 234 in Figure 7 middle). This simulation does not include any other inflow or PMP input. The channel roughness value (n-value) of 0.018 chosen by the licensee in the FHRR was used throughout this simulation. The result of simulation (Figure 7 bottom-right) indicates significant change in flood levels by different breach scenarios. As the simulated flood levels on the plant site would exceed the plant grade, the staff determined to simulate HEC-RAS with an unsteady option to determine the onsite flood levels accurately.

8.2. Unsteady Simulation[14]

For this simulation, the staff adopted the channel configuration for the previous steady simulation, but added more cross sections using the HEC-RAS cross section interpolation option in order to ensure the stability of unsteady simulations (see Figure 8 top left). In addition, the staff modified the SE upstream cross sections so that these sections represent SE1 and SE2 subbasins correctly. That is, the HEC-RAS model was set up so that the breach flows entered as an upstream boundary condition was divided into two directions: discharge to the east and flow to the SE1 and SE2 basin temporary but discharge later to the east. In this framework, a portion of breach flow on the south east of the plant site is stored temporary on the SE1 and SE2 subbasins by the backwater effect as was shown on the HEC-RAS output.

The staff simulated the four postulated breach scenarios presented in the Subsection 7.2: three gap scenarios and one crack-piping scenario. For each scenario, the staff used the simulated breach hydrograph as an upstream as an upstream inflow to the SE tributary channel upstream.

Table 7 summarizes the result of the HEC-RAS unsteady simulations. For Scenario 4 which is the most plausible scenario, the staff identified that about 27 percent of the upstream inflow or 6 percent of the total SWR breach outflow would be stored in the SE1 and SE2 subbasins temporarily. This storage volume would create a maximum flood level of 273.76 ft msl. This estimated flood level exceeds the plant grade but is lower than the licensee estimated maximum LIP flood level of 274.5 ft msl at the power block area. The Figure 7 bottom shows the simulated stage hydrograph at the uppermost section of the SE2 subbasin. Table 8

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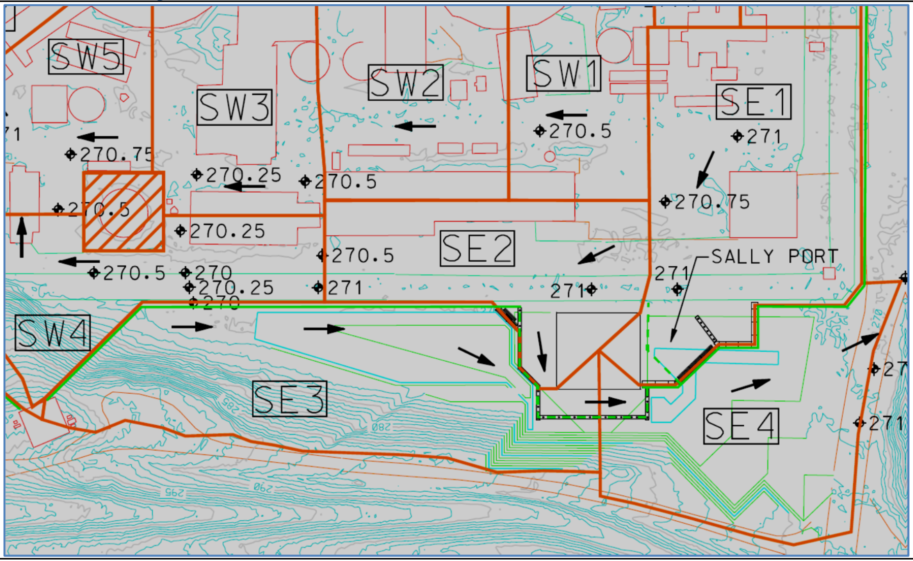
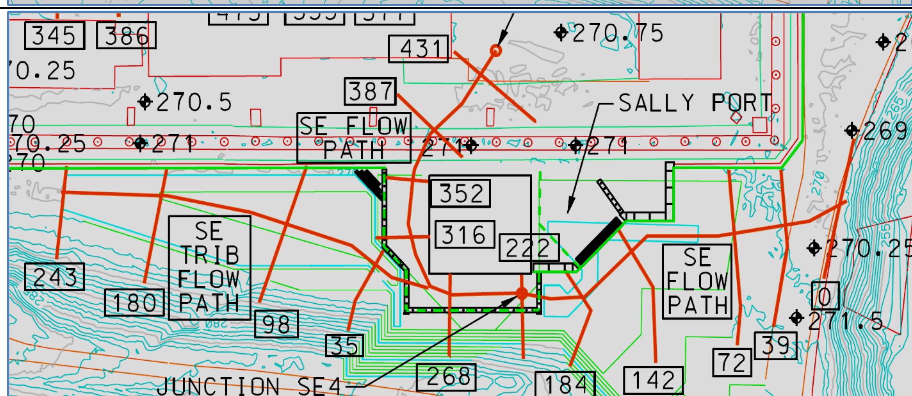
summarizes initial and attenuated breach peak flow rates and time to peaks at different channel locations.

Figure 7. HEC-RAS steady simulation of breach outflows at the plant site.

SE subbasins (from the FHRR Figure 2.1-1)

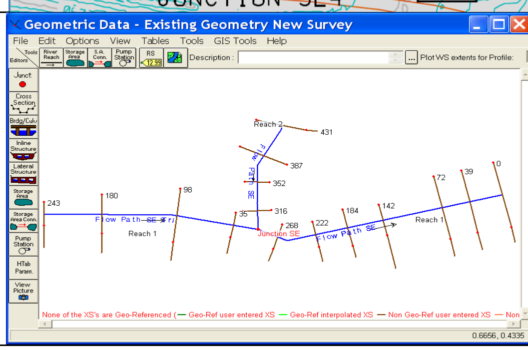


Drainage pathways (from the FHRR Figure 2.1-4)



Left: channel configuration.

Right: a summary of HEC-RAS simulations



Scenario	Peak Breach Outflow, cfs	Simulated Flood Level, ft msl
1	186	272.91
2	472	274.15
3	734	275.04
4	373	274.76

It should be noted that the elevations on the NA topographic map (Figure 1(b)) are with respect to NAVD 88, while the water surface elevation for flooding due to local intense precipitation (274.5 ft. msl) is with respect to NVGD 29 (see Section 2.1 on page 2.1-1 of NA FHRR). If the correction of +0.86 ft. was not added to either the HEC-RAS cross-section elevations or the simulated water surface elevation (not both, of course), then the maximum flood level of 273.76 ft. msl should be 274.62 ft. msl.

Table 7. Result of the HEC-RAS steady simulation of breach outflows at the plant site.

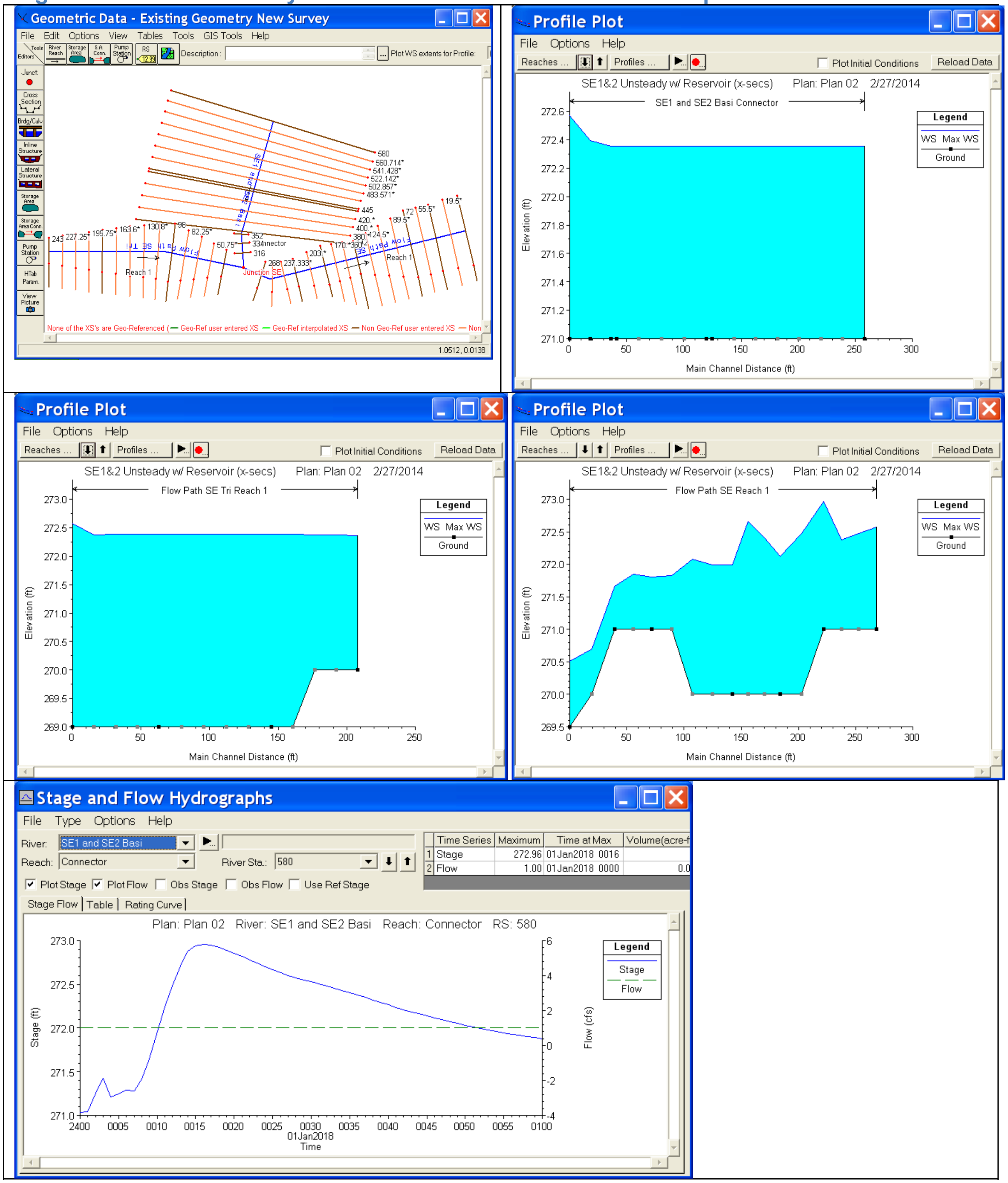
Scenario	Simulated Flood Level, ft msl	Total Inflow, ft^3	Water Stored in SE1 & SE2, ft^3	Fraction of Storage to Emer. Dike Breach Flow , %	Fraction of Storage to Total Breach Flow, %
1	272.91	280000	123195	44	4
2	274.15	551000	203175	37	7
3	275.04	897000	260580	29	9
4	273.76	649000	178020	27	6

Notes: where the total storage area for SE1 & SE2 is approximately about 64 500 ft2, and the bottom elevation .of the subbasins is 271 ft msl

Table 8. Summary of breach peak flows and time to peak at different locations.

Location	Breach Peak Flow, cfs	Breach Peak Time, minutes
reservoir dike Downstream	2,780	11
Encroachment Section	2,715	12
Emergency Dike Breach Downstream	373	21

Figure 8. HEC-RAS steady simulation of breach outflows at the plant site.



9. Conclusion

The NRC staff performed a hierarchical hydraulic modeling to evaluate the effects of potential dike failure floods from the Service Water Reservoir on the plant site. The staff also evaluated the capacity of the emergency dike and intercepting channel system to determine whether the system has a sufficient diversion capacity without impacting the plant site or not. The hydraulic modeling was intended to supplement the staff's review of the FHRR.

The modeling includes breach parameter estimation, uniform flow calculation, and HEC-RAS flood steady and unsteady routing on the intercepting channel and onsite drainage system. The staff assumed that both SWR dike and emergency dike would be failed simultaneously by either seismic or piping event. The followings are a summary of the major finding from the hydrologic modeling:

- Using empirical regression equations, the staff estimated SWR dike breach parameters as: average breach width of 60 ft, breach time of 0.46 hours, and breach peak outflow of 10,319 cfs.

- The staff confirmed through a HEC-RAS unsteady simulation that the emergency dike and intercepting channel has enough capacity to divert potential breach outflows without overtopping.

- Using a HEC-RAS unsteady simulation with a postulated dike failure scenario caused by a piping or seismic mode, the staff obtained a peak breach outflow through the SWR dike of 2,800 cfs and a peak time of 12 minutes after the initiation of a breach. These model estimates are much more accurate than those by the regression equations.

- The storage capacity of the SWR is 3,033,000 ft^3. The staff estimated based on a HEC-RAS simulation that about 21 per cent of the SWR storage volume wound be discharged through the emergency dike breach gap. About 27 per cent of the emergency dike breach outflow will be retained temporally in the southeast corner of the plant site (so called the SE subbasins), creating the maximum flood level of 273.76 ft msl, which exceed the plant grade but lower than the licensee-estimated LIP flood level of 274.5 ft msl.

- Therefore, the licensee needs a seismic qualification to ensure that the SWR dike will not fail.

In this dike failure flood analysis, the staff used the following conservative assumptions:

- The piping breach parameters (e.g., breach width and time) for the emergency dike were estimated under the assumption that the embankment dike soil materials are saturated as in the case of the SWR dike failure. For unsaturated dike condition, actual piping growth rate and final breach width would be much less than the estimates by this assumption.

- Emergency dike breach outflow is assumed to transfer instantaneously from the emergency dike breach spot to the onsite drainage channel on the plant site without attenuation (approximately 150 ft distance), giving a conservative peak flow rates and inundation levels at the plant site.

- The HEC-RAS simulation considers the storage in the southeast subbasins only: however actual breach flood wave on the plant site would be spread further the north, resulting in a smaller flood level at the plant site.

- The analysis assumes no loss from infiltration, retention, and evaporation losses conservatively.