Progress Energy

Serial: NPD-NRC-2009-056
April 1, 2009

10CFR52.79

U.S. Nuclear Regulatory Commission
Attention: Document Control Desk
Washington, D.C. 20555-0001

SHEARON HARRIS NUCLEAR POWER PLANT, UNITS 2 AND 3
DOCKET NOS. 52-022 AND 52-023
SUPPLEMENT 1 TO RESPONSE TO REQUEST FOR ADDITIONAL INFORMATION LETTER NO. 023 RELATED TO PROBABLE MAXIMUM FLOOD ON STREAMS AND RIVERS

References:

1. Letter from Manny Comar (NRC) to James Scarola (PEC), dated September 26, 2008, "Request for Additional Information Letter No. 023 Related to SRP Section 02.04.03 for the Harris Units 2 and 3 Combined License Application"

2. Letter from James Scarola (PEC) to U. S. Nuclear Regulatory Commission (NRC), dated October 31, 2008, "Response to Request for Additional Information Letter No. 023 Related to Probable Maximum Flood on Streams and Rivers," Serial: NPD-NRC-2008-054

Ladies and Gentlemen:

Progress Energy Carolinas, Inc. (PEC) hereby submits a supplemental response to the Nuclear Regulatory Commission's (NRC) request for additional information provided in the referenced letter (Reference 1).

A revised response to the NRC request is addressed in the enclosure. The enclosure also identifies changes that will be made in a future revision of the Shearon Harris Nuclear Power Plant Units 2 and 3 application.

If you have any further questions, or need additional information, please contact Bob Kitchen at (919) 546-6992, or me at (919) 546-6107.

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

Executed on April 1, 2009.

Sincerely,

Garry D. Miller
General Manager
Nuclear Plant Development

Enclosure

Progress Energy Carolinas, Inc.
P.O. Box 1551
Raleigh, NC 27602

D084
NRO

cc : U.S. NRC Director, Office of New Reactors/NRLPO
U.S. NRC Office of Nuclear Reactor Regulation/NRLPO
U.S. NRC Region II, Regional Administrator
U.S. NRC Resident Inspector, SHNPP Unit 1
Mr. Manny Comar, U.S. NRC Project Manager

bc : Robert Kitchen, Manager-Nuclear Plant Licensing
Chris Kamilaris, Director-Fleet Support Services
Tillie Wilkins, NPD-Licensing
John O'Neill, Jr. (Pillsbury Winthrop Shaw Pittman, LLP)
A. K. Singh (Sargent & Lundy, LLC)
Jim Steele (CH2M HILL)
John Archer (WorleyParsons)
File: NGG-NPD (Dawn Bisson)
NPD Document Control Inbox (Records: Correspondence)

Shearon Harris Nuclear Power Plant Units 2 and 3
Supplement 1 to Response to NRC Request for Additional Information Letter No. 023
Related to SRP Section 02.04.03 for the Combined License Application,
dated September 26, 2008

NRC RAI #	Progress Energy RAI #	Progress Energy Response
02.04.03-4	H-0435	Revised response enclosed – see following pages

NRC Letter No.: HAR-RAI-LTR-023

NRC Letter Date: September 26, 2008

NRC Review of Final Safety Analysis Report

NRC RAI #: 02.04.03-4

Text of NRC RAI:

Based on discussions with US Army Corps of Engineers personnel, the NRC staff have determined that ETL 1110-2-221 "Wave Runup and Wind Setup on Reservoir Embankments" has been superceded by guidance found in EM 1110-2-1420 "Hydrologic Engineering Requirements for Reservoirs" and guidance found in the Coastal Engineering Manual (EM 1110-2-1100). The staff requests the applicant to show how their current methodology results in a conservative estimate of wind-wave effects.

PGN RAI ID #: H-0435

PGN Response to NRC RAI:

Subsequent to Progress Energy's initial response to this RAI, a follow-up meeting was requested by NRC to facilitate a better understanding of NRC's request concerning the PMF estimate. The meeting was held in Raleigh, North Carolina, on February 10 and 11, 2009, and was attended by representatives of Progress Energy and NRC. During the meeting, Progress Energy summarized the methodology and approach that was used for the development of the PMF estimate for the site and described why that estimate was considered to be highly conservative. As a result of the meeting, Progress Energy has revised the PMF analysis to include the use of the U.S. Army Corps of Engineers' *Coastal Engineering Manual* to account for wind setup and wave run-up, adding additional conservatism to the PMF analysis for HAR 2 and HAR 3. The information provided in this response is intended to supersede and replace our previous response to this RAI, which was submitted to NRC by letter dated October 31, 2008 (Serial NPD-NRC-2008-054).

The effect of wind setup and wave run-up for a worst-case PMF event has been evaluated in the vicinity of Safety-Related Structures at the HAR site, as well as at the existing Main and Auxiliary Dams. A discussion of each of these is provided below.

<u>Wind Setup and Wave Runup - Impacts in the Vicinity of HAR Safety-Related Structures</u>
As discussed in FSAR Subsection 2.4.2.2, Flood Design Considerations, safety-related structures and facilities for the HAR site are protected against floods and flood waves caused by probable maximum events, such as the probable maximum flood (PMF) and the probable maximum hurricane (PMH). Coincident Wind Wave Activity was evaluated at two safety-related locations: HAR 2 and HAR 3. For these two locations, the U.S. Army Corps of Engineers' *Coastal Engineering Manual, Engineer Manual 1110-2-1100 (Part II)* (*U.S. Army Corps of Engineers, 2002, Coastal Engineering Manual, Part II, Chapter 2: Meteorology and Wave Climate. Engineer Manual 1110-2-1100, U.S. Army Corps of Engineers, Washington, D.C.*) was strictly followed to determine wave runup.

In order to determine wind setup and wave runup for HAR 2 and HAR 3, the following data was required:

- Water body bathymetry data
- Critical fetch distances
- Over-wind speed averaged for an appropriate duration
- Site characteristics such as type and material of protection, and slope

Bathymetry Data

Progress Energy performed detailed bathymetric surveys to establish the current geometry of the Main Reservoir with thousands of depth-to-bottom measurements collected during the study. These data were compiled into a single GIS point coverage. ArcGIS three-dimensional (3-D) analyst Kriging sampling interpolation was used to generate a 3-D surface from the mass point data. For further detail on the processing of bathymetric data, refer to RAI 02.04.03-1 (Serial NPD-NRC-2009-050).

The spatial distribution of water depths in the Main Reservoir corresponding to the maximum PMF level of 256.99 feet NGVD29 was determined using output from the HEC-RAS model. A summary of these depths is presented in **Figure 1** and **Table 1**. For wind and wave calculation purposes, the 75th percentile of the water depth (76 feet) was used.

Figure 1: Histogram of Main Reservoir Depths Corresponding to the Maximum PMF Level

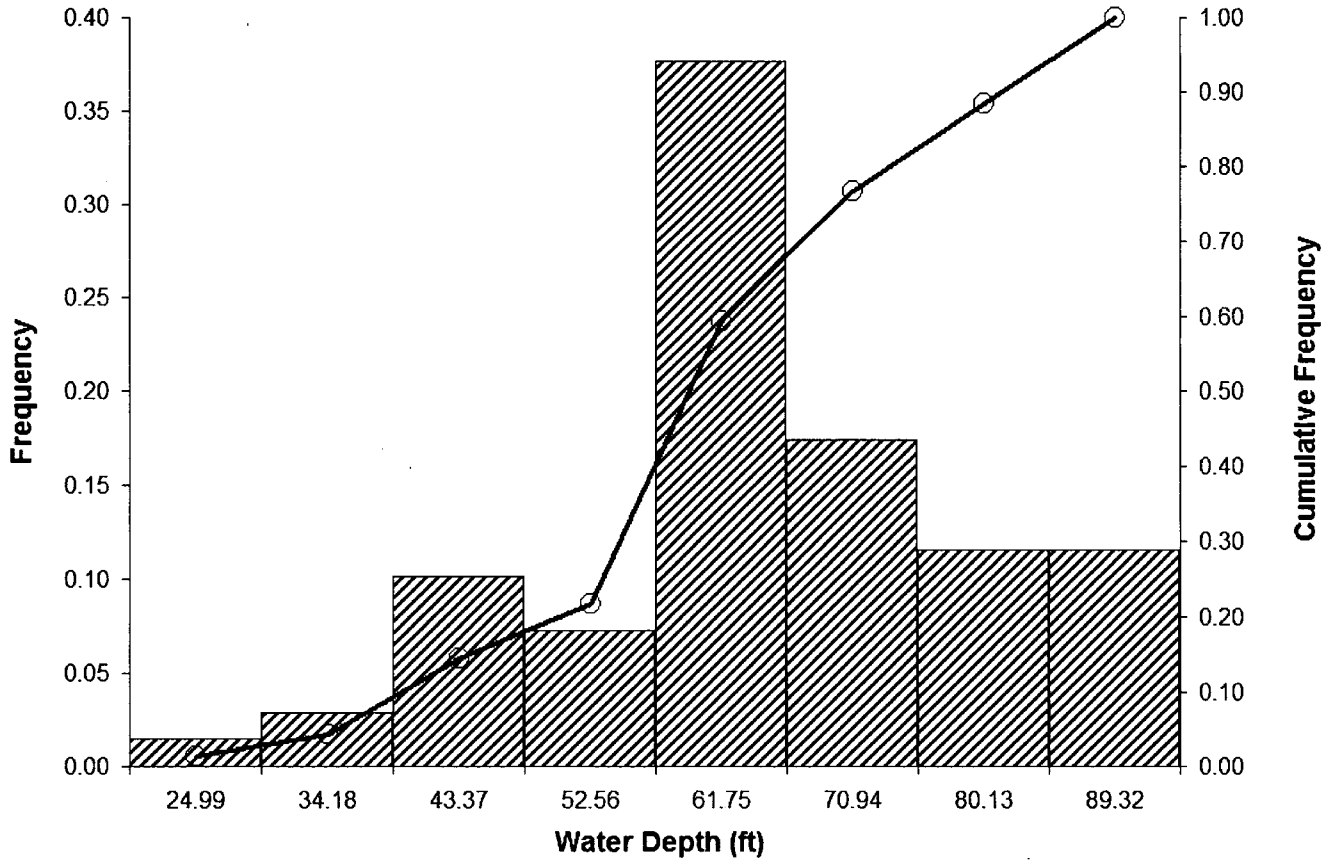


Table 1: Histogram Data Main Reservoir Depths

Depth (ft)	Frequency	Cumulative Frequency
25.0	0.014	0.01
34.2	0.029	0.04
43.4	0.101	0.14
52.6	0.072	0.22
61.8	0.377	0.59
70.9	0.174	0.77
80.1	0.116	0.88
89.3	0.116	1.00

Statistical Characteristics

Mean	60.5	ft
Standard Deviation	14.8	ft
CV	0.24	NA
75th percentile	76	ft

Fetch Distances

Fetch is the length of water surface exposed to wind during the generation of waves. Fetch is an important characteristic of open water because longer fetch can result in larger wind-generated waves. For this study, the most relevant directions for wind-wave activity are from the east and west. There are substantial intervening land masses to the north and south of HAR 2 and HAR 3 that exclude these directions from significant wind-wave activity.

According to *EM 1110-2-1100* straight line fetch distances were used in wave runup calculations. Thus, straight line fetch distances were determined using the site topographic and reservoir bathymetry data as shown in **Figure 2** and **Figure 3**. **Table 2** and **Table 3** provide the overland and overwater fetch distances from HAR 2 and HAR 3. The critical fetch distances range from 0.85 to 0.93 mile.

Table 2: Fetch Distances for HAR 2

Id	Fetch Dist (mile)
1	0.93*
2	0.88
3	0.87
4	0.40
5	0.50
6	0.50
7	0.50

*Critical Fetch Distance = 0.93 mi

Table 3: Fetch Distances for HAR 3

Id	Fetch Dist (mile)
1	0.85*
2	0.72
3	0.61
4	0.64
5	0.55

*Critical Fetch Distance = 0.85 mi

Figure 2: Direct Fetch for the HAR-2 Safety Related Structures

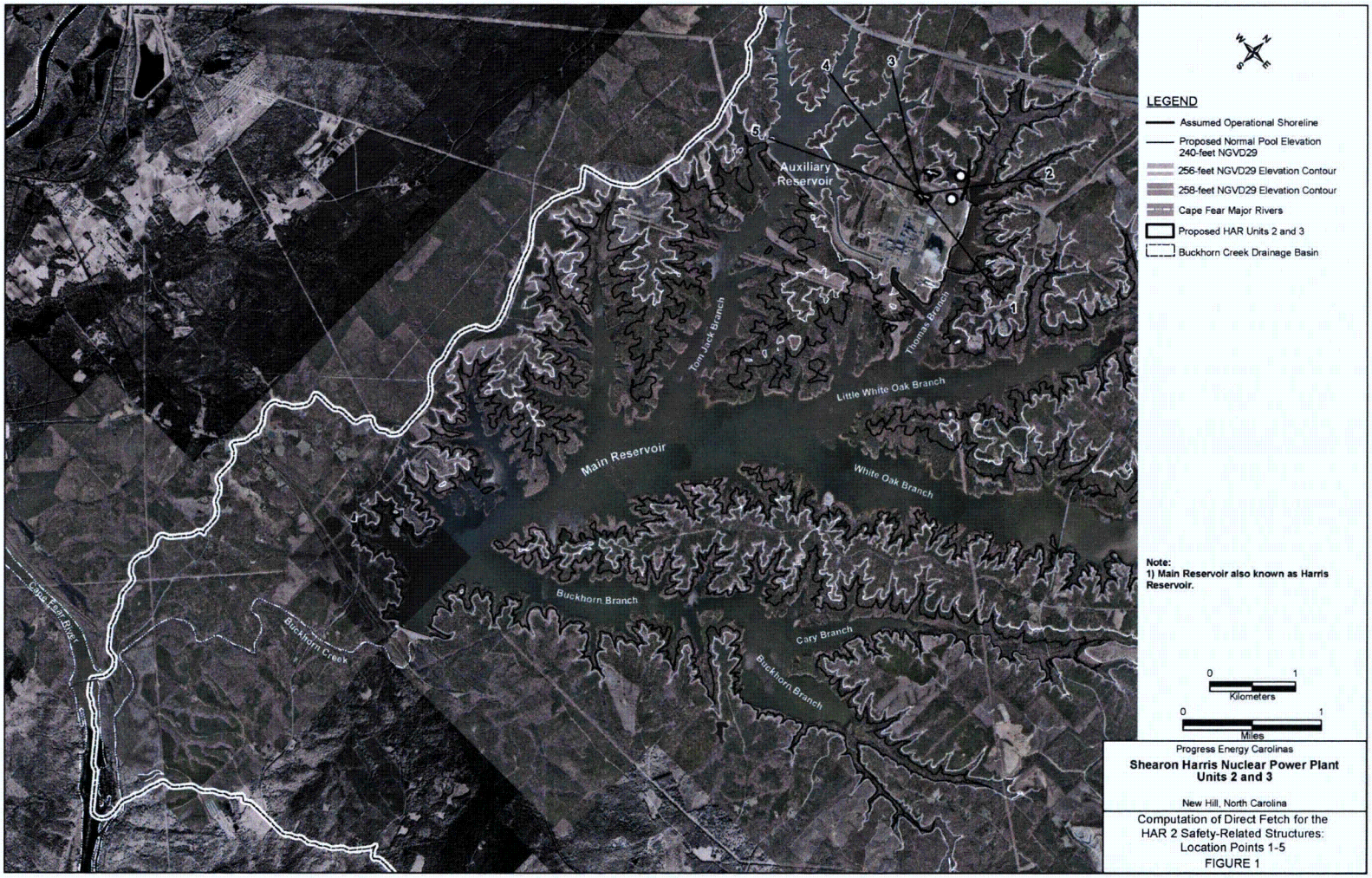
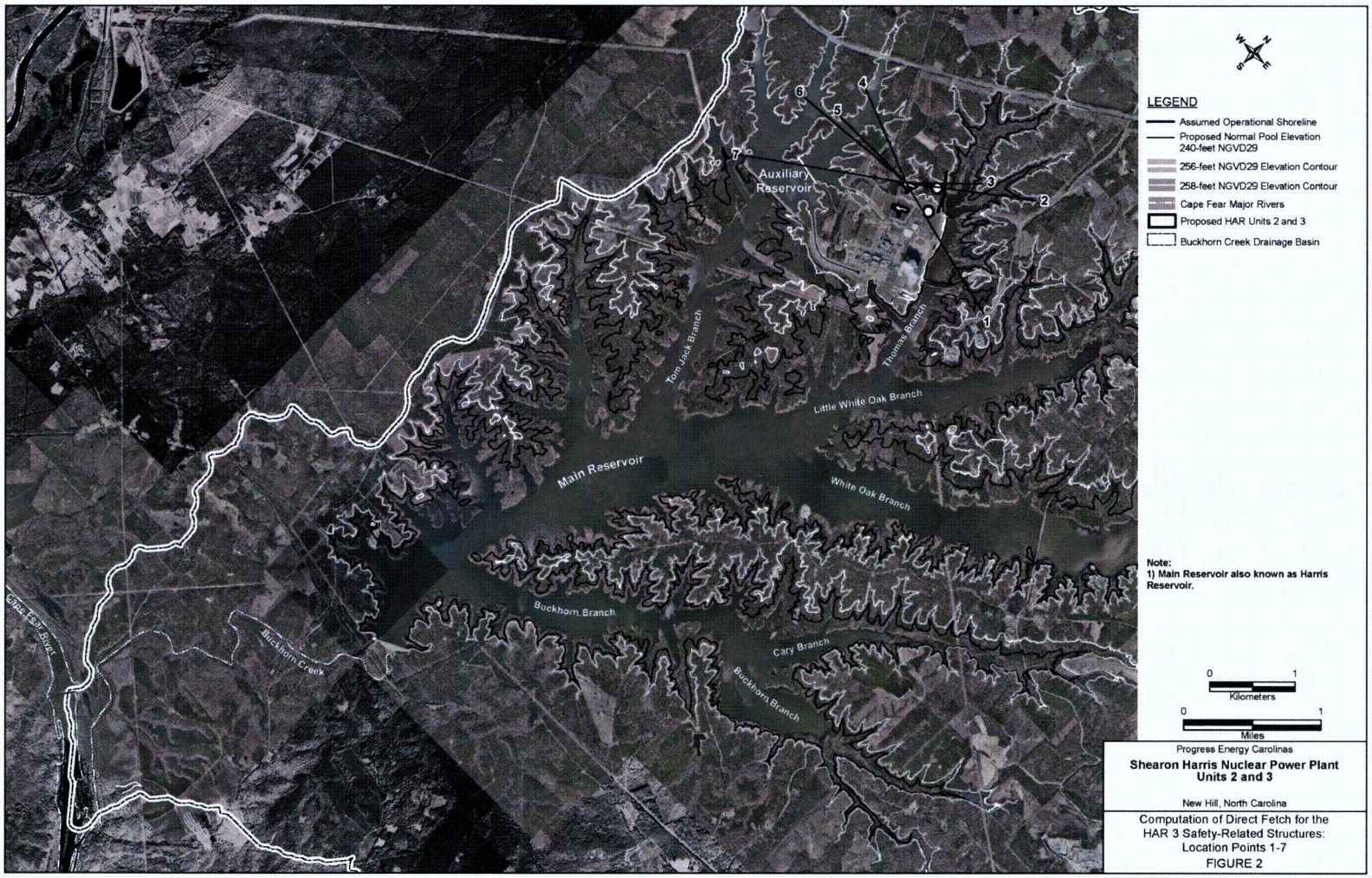


Figure 3: Direct Fetch for the HAR-3 Safety Related Structures



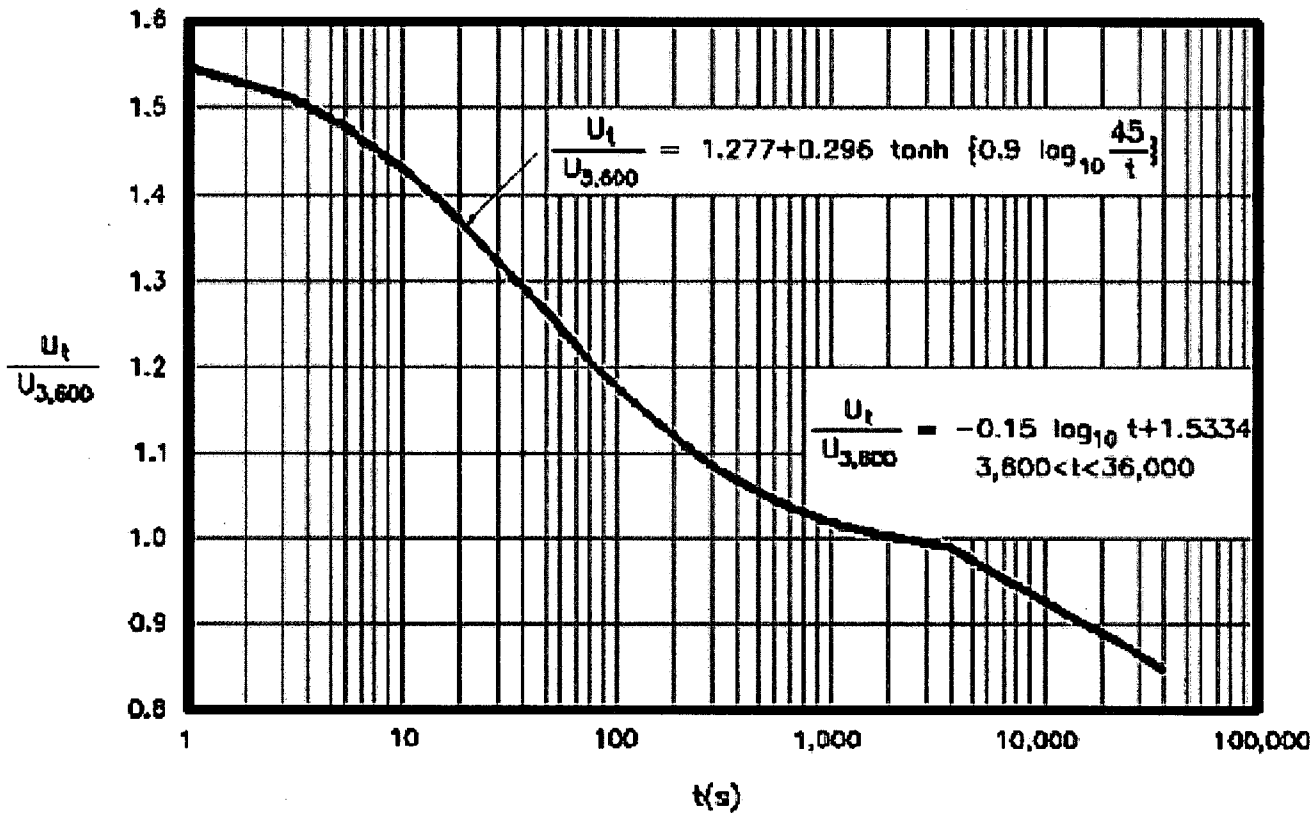
Over-Water Wind Speed

The 5-second peak gust speed was obtained from the *National Climatic Data Center's (NCDC) wind data records for the Raleigh Airport, NC from 1930-96* (*FSAR Reference 2.4-233*). According to this document, 5-second peak gust wind speed at the Raleigh Airport is 67 mph. Before using this wind speed in the calculation of wave runup, several adjustments were applied following the procedure outlined in the *EM 1110-2-1100 (Part II)*. A step-by-step procedure is described below:

1. Standard measurements should be collected at 10 meters above ground surface. Since the wind speed at the Raleigh Airport was measured at this height, no adjustment needs to be applied.

2. Using the 5-second peak gust of 67 mph (*FSAR Reference 2.4-233*) and Figure II-2-1 (**Figure 4**) of *EM 1110-2-1100 (Part II)*, the 1-hour wind speed was calculated as 45.2 mph.

Figure 4: Ratio of Wind Speed of any Duration, U_t to the 1-Hour Wind Speed U_{3600}



3. Overwater wind speeds at various locations were then determined by applying a correction for transition from land to water. This correction factor was determined using Figure II-2-7 (**Figure 5**) of *EM 1110-2-1100 (Part II)*. (Note: the HAR site is approximately 140 miles from the coastal line, hence, Figure II-2-7 (**Figure 5**) of *EM 1110-2-1100 (Part II)* is applicable.)

According to Figure II-2-7 (**Figure 5**), the correction factor R_L is given as follows:

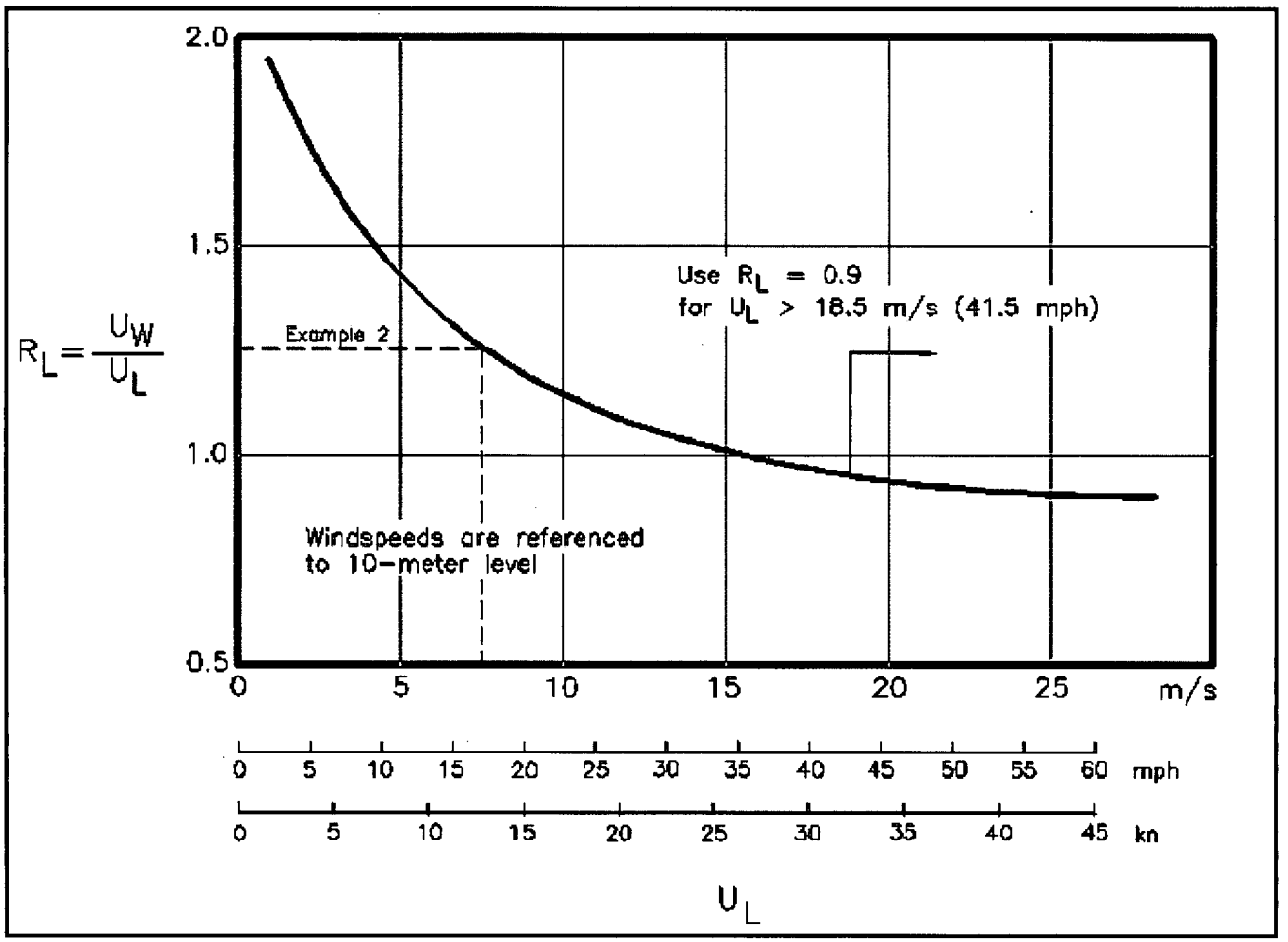
$$U_W = R_L * U_L \quad (1)$$

where:

U_W is the overwater wind speed,
U_L is the overland wind speed, and
R_L is a correction factor which is equal to 0.9 for U_L > 41.5 mph.

In this case, the overland wind speed, U_L, is 45.2 mph (20.1 m/s). As such, the correction factor, R_L, is equal to 0.9. However, in an effort to be conservative, a correction factor, R_L, equal to 1.0 was used for this analysis.

Figure 5: Ratio R_L of Wind Speed Overwater U_W to Wind Speed Overland U_L as a Function of Wind Speed Overland U_L (after Resio and Vincent (1977)



4. The wind speed is now corrected according to the appropriate averaging duration. When wind occurs with essentially constant direction over a fetch for sufficient time to achieve steady-state, fetch–limited values, simplified wave predictions can provide accurate estimates of wave conditions. The time required to accomplish fetch-limited wave development for short fetches was calculated as follows:

$$t_{x,u} = 77.23 \frac{X^{0.67}}{u^{0.34} g^{0.33}} \qquad (2)$$

where:

$t_{x,u}$ is the time required for waves crossing a fetch of length x under a wind of velocity u to become fetch-limited.

The resulting averaging time interval, $t_{x,u}$ was then used in conjunction with Figure II-2-1 of *EM 1110-2-1100 (Part II)* (**Figure 4**) in order to determine the appropriate wind speed for various locations. **Table 4** presents calculated corrections of wind averaging intervals for various fetch lines directed toward HAR 2 and HAR 3.

Table 4: Correction for Wind Averaging Interval

Location	Line ID	St. Line Fetch, X (mi)	St. Line Fetch, X (km)	t(X,U) (sec)	Correction Factor	Wind Speed, Ut (m/s)
HAR 2	1	0.85	1.36	1647	1.01	20.4
	2	0.72	1.16	1481	1.02	20.4
	3	0.61	0.98	1325	1.02	20.5
	4	0.64	1.02	1360	1.02	20.5
	5	0.55	0.88	1234	1.02	20.5
HAR 3	1	0.93	1.49	1752	1.01	20.3
	2	0.88	1.41	1689	1.01	20.4
	3	0.87	1.39	1673	1.01	20.4
	4	0.40	0.63	988	1.03	20.7
	5	0.50	0.80	1157	1.02	20.6
	6	0.50	0.80	1158	1.02	20.6
	7	0.50	0.80	1155	1.02	20.6

Site characteristics such as type and material of protection, and slope

HAR 2 and HAR 3 are surrounded by the Thomas Creek Branch of the Main Reservoir on the East side and by the Auxiliary Reservoir on the West side. FSAR Figure 2.4.1-205 shows the planned site drainage plan and indicates that HAR 2 and HAR 3 will have permeable natural land area between the developed site and the water bodies. Using site specific topographic and bathymetry data, the land slope adjacent to the Thomas Creek Branch of the Main Reservoir and Auxiliary Reservoir were determined; these slopes were 0.087 and 0.126 for the east and west sides of HAR 2 and HAR 3, respectively.

Wave Runup

Having determined the estimate of winds for wave prediction, wave runup for various fetch lines directed toward HAR 2 and HAR 3 were calculated according to the step-by-step procedure given in the *EM 1110-2-1100 (Part VI)* (*U.S. Army Corps of Engineers, 2006, Coastal Engineering Manual, Part VI, Chapter 5: Fundamentals of Design, Engineer Manual 1110-2-1100, U.S. Army Corps of Engineers, Washington, D.C.*). A brief description is given below:

1. Using the previously determined fetch lengths and wind speeds, estimates of significant wave heights were obtained using the deepwater nomogram for the fetch limited wave heights given in the *EM 1110-2-1100 (Part II)* (**Figure 6**).

2. Similarly, estimates of peak wave periods were obtained using the deepwater nomogram for the fetch limited wave periods given in the *EM 1110-2-1100 (Part II)* (**Figure 7**).

Figure 6: Fetch-Limited Wave Heights

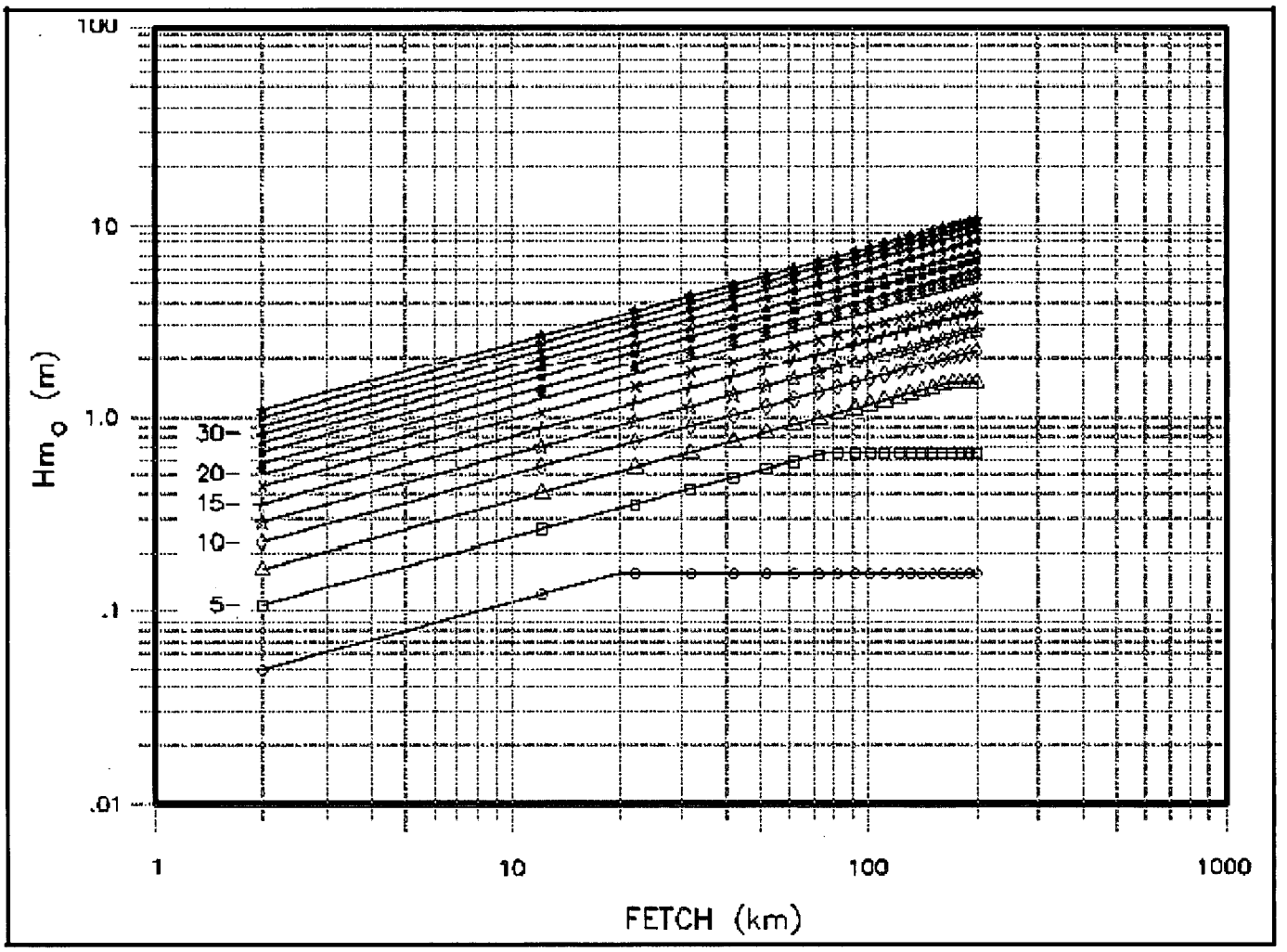
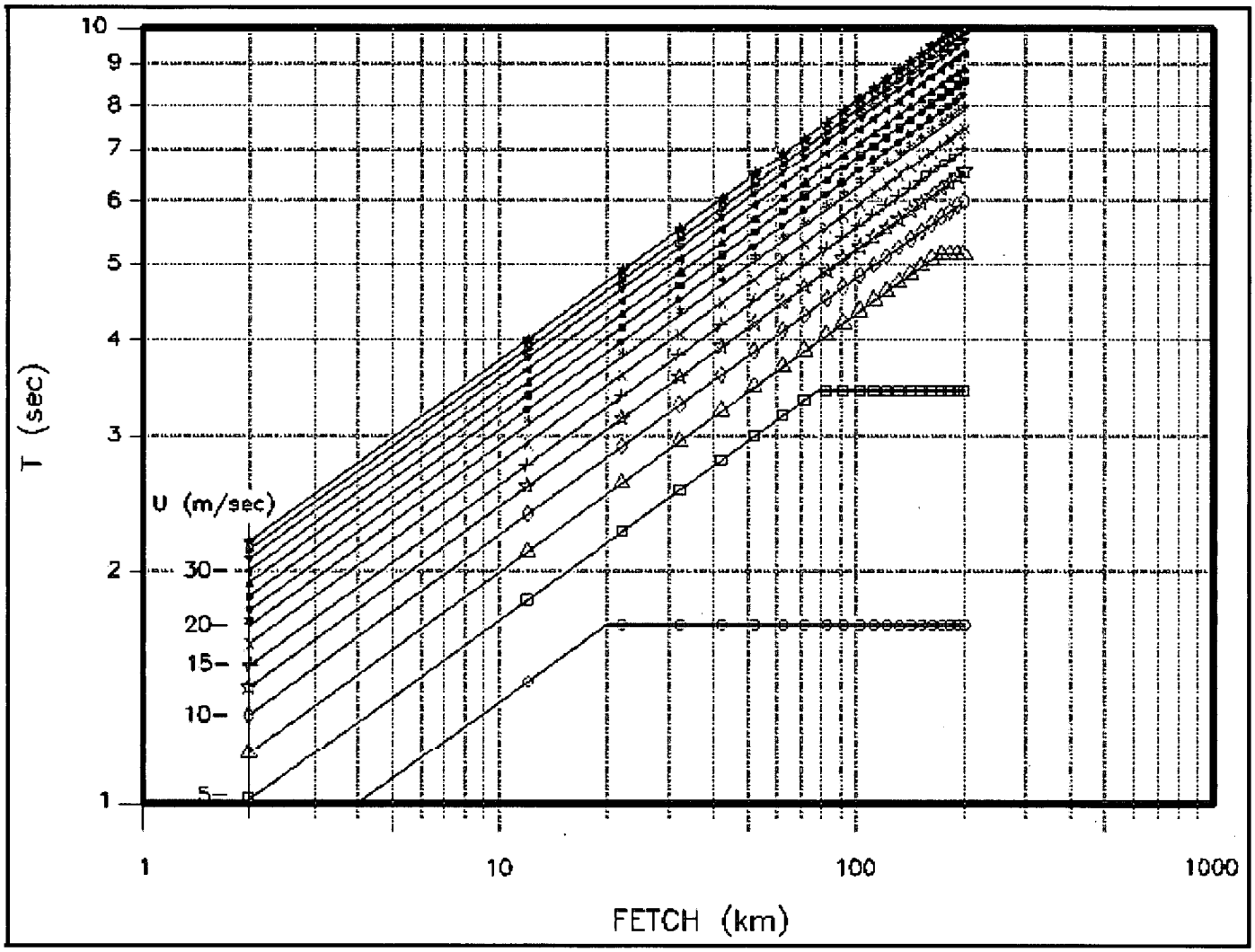


Figure 7: Fetch-Limited Wave Periods (Wind Speed in Increments of 2.5 m/s)



3. The peak wave periods for various fetch lines calculated above were compared with the shallow-water limit. According to *EM 1110-2-1100 (Part II)*, the shallow-water limit is given by the following equation:

$$T_P \approx 9.78\left(\frac{d}{g}\right)^{\frac{1}{2}} \qquad (1)$$

Where:

T_p = the limiting wave period in seconds,
d = the water depth in meters, and
g = the gravitational acceleration in meter/sec^2.

If the predicted peak wave period for a given fetch line is greater than the limiting value, then the predicted wave period is reduced to the limiting wave period. Conversely, if the predicted wave period was less than the limiting value, the predicted deepwater wave period was retained and used for further calculations.

4. Wave runup was calculated using the wave runup equation on permeable slopes given in Chapter 5 of *EM 1110-2-1100(Part VI)*. According to the CEM (2006), the runup equation for various levels of percentage exceedances is given as:

$$R_{ui\%}/H_S = A\xi_{om} \quad for \;\; 1.0 < \xi_{om} \leq 1.5 \tag{2a}$$

$$R_{ui\%}/H_S = B\xi_{om}^{C} \quad for \;\; 1.5 < \xi_{om} \leq (D/B)^{1/C} \tag{2b}$$

$$R_{ui\%}/H_S = D \; for \;\; (D/B)^{1/C} < \xi_{om} \leq 7.5 \tag{2c}$$

Where, ξ_{om} is the surf-similarity parameter for irregular waves defined as:

$$\xi_{om} = \frac{\tan\alpha}{\sqrt{s_{om}}} \tag{3}$$

In which S_{om} is the fictitious wave steepness defined as the ratio between the statistical wave height at the structure and representative deepwater wavelengths or as follows:

$$s_{om} = \frac{H_s}{L_{om}} = \frac{2\pi}{g}\frac{H_s}{T_m^2} \tag{4}$$

Where:

H_s = significant wave height of incident waves at the toe of the structure,
T_m = mean wave period, and
T_P = wave period corresponding to the peak of the wave spectrum.

The coefficients A through D in Equations (2a, 2b, and 2c) for runup of irregular head-on waves on impermeable and permeable rock armored slopes are given in **Table 5**.

Table 5: Fetch Distances

Percent	A	B	C	D
0.1	1.12	1.34	0.55	2.58
2	0.96	1.17	0.46	1.97
1	1.04	1.26	0.51	2.29
Significant	0.72	0.88	0.41	1.35

Using the steps 1 through 4 for various fetch lines, runup was calculated. **Table 6** presents the runup results.

Table 6: Runup Calculation for the HAR 2 and HAR 3 Sites

Location	Line ID	Wind Velocity (m/s)	St. Line Fetch, X (km)	Hm0 (m)	Predicted Peak Wave Period, Tp (sec)	Slope = tan(alpha)	Deepwater Wave Steepness, s0	Iribarren Number	Significant Runup, Rus (ft)	Max Runup (0.1%), Ru (ft)
HAR 2	1	20.37	1.36	0.42	1.73	0.09	0.09	0.29	0.29	0.45
	2	20.43	1.16	0.39	1.64	0.09	0.09	0.29	0.26	0.41
	3	20.49	0.98	0.36	1.56	0.13	0.10	0.41	0.35	0.54
	4	20.47	1.02	0.37	1.58	0.13	0.09	0.41	0.36	0.55
	5	20.53	0.88	0.34	1.50	0.13	0.10	0.40	0.33	0.51
HAR 3	1	20.34	1.49	0.44	1.78	0.09	0.09	0.29	0.30	0.47
	2	20.36	1.41	0.43	1.75	0.09	0.09	0.29	0.29	0.46
	3	20.36	1.39	0.43	1.74	0.09	0.09	0.29	0.29	0.45
	4	20.67	0.63	0.29	1.35	0.13	0.10	0.39	0.27	0.42
	5	20.57	0.80	0.33	1.46	0.13	0.10	0.40	0.31	0.48
	6	20.57	0.80	0.33	1.46	0.13	0.10	0.40	0.31	0.48
	7	20.57	0.80	0.33	1.46	0.13	0.10	0.40	0.31	0.48

Note: Full pool water depth = 23.2 m and corresponding limiting wave period = 15.03 sec.

Wind Setup

When wind blows over a water body, it exerts a horizontal stress on the water surface in the wind direction. In an enclosed water body, this wind effect results in a piling up of water at the leeward end and a lowering of water level at the windward end. This effect is called wind setup. According to *EM 1110-2-1420* (*U.S. Army Corps of Engineers, 1997, Engineering and Design, Hydrologic Engineering Requirements for Reservoirs, Chapter 15: Dam Freeboard Requirements, Engineer Manual 1110-2-1420, U.S. Army Corps of Engineers, Washington, D.C.*), the wind setup in lakes and reservoirs can reasonably be estimated using the Zeider Zee equation given as:

$$S = \frac{U^2 X}{1400 d} \qquad (5)$$

Where:

S = the setup (ft) above the Stillwater level,
U = the wind speed (mph),
F = the fetch length (mile), and
d = the water depth corresponding to the PMF level.

Table 7 presents the setup calculation using the Equation (5).

Table 7: Setup Calculation for the HAR 2 and HAR 3 Sites

Location	Line ID	Wind Velocity, U (mph)	St. Line Fetch, X (mi)	Depth, d (ft)	Setup, S (ft)
HAR 2	1	45.84	0.85	76.00	0.02
	2	45.96	0.72	76.00	0.01
	3	46.10	0.61	76.00	0.01
	4	46.06	0.64	76.00	0.01
	5	46.19	0.55	76.00	0.01
HAR 3	1	45.77	0.93	76.00	0.02
	2	45.81	0.88	76.00	0.02
	3	45.82	0.87	76.00	0.02
	4	46.52	0.40	76.00	0.01
	5	46.28	0.50	76.00	0.01
	6	46.28	0.50	76.00	0.01
	7	46.28	0.50	76.00	0.01

Overall PMF Elevation

In order to determine the PMF elevation coincident with wind-wave activity at HAR 2 and HAR 3, the obtained values of stillwater elevation, wave runup, and wind setup for various fetch lines were added together. **Table 8** presents the overall PMF elevations. From this table, the maximum PMF elevations associated with HAR 2 and HAR 3 are 257.47 and 257.49 feet NGVD29, respectively.

Table 8: Overall PMF Elevation at the HAR 2 and HAR 3 Sites

Location	Line ID	Stillwater	Max Runup (0.1%), Ru (ft)	Setup (ft)	Overall PMF (ft)
HAR 2	1	257.00	0.45	0.02	257.47
	2	257.00	0.41	0.01	257.42
	3	256.50	0.54	0.01	257.05
	4	256.50	0.55	0.01	257.07
	5	256.50	0.51	0.01	257.02
HAR 3	1	257.00	0.47	0.02	257.49
	2	257.00	0.46	0.02	257.48
	3	257.00	0.45	0.02	257.47
	4	256.50	0.42	0.01	256.93
	5	256.50	0.48	0.01	256.99
	6	256.50	0.48	0.01	256.99
	7	256.50	0.48	0.01	256.99

Note: 257.0 and 257.25 feet NGVD29 are the maximum Stillwater PMF elevations for the Main and Auxiliary Reservoirs.

Wind Setup and Wave Runup - Impacts at the Existing Main and Auxiliary Dams

The operation of HAR 2 and HAR 3 will not be dependent on the existing Main and Auxiliary Dams from a safety standpoint, so a detailed analysis of wind-wave impacts at the dams has not been performed. However, during the course of the analysis described herein it was noted that the impacts of wave setup and wave run-up could have a potential impact at the Main and Auxiliary Dams due to the fetch distance between them. Progress Energy will perform a comprehensive evaluation of the potential impact of wind-wave interaction on both dams to determine if any mitigation will be required. This evaluation will be performed and completed by the end of the third quarter of 2009.

Associated HAR COL Application Revisions:

The information provided in this response will be summarized and incorporated into a future revision of FSAR Subsection 2.4.3.

Attachments/Enclosures:

None.