## 3.7 Ice-Induced Flooding

Ice jams and ice dams can form in rivers and streams adjacent to a site and may lead to flooding by two mechanisms (NRC, 2011):

- Collapse of an ice jam or a dam upstream of the site can result in a dam breach-like flood wave that may propagate to the site, and
- An ice jam or a dam downstream of a site may impound water upstream of itself, thus causing a flood via backwater effects.

This section summarizes the Ice-Induced Flooding evaluation performed in AREVA Calculation No. 32-9227011-000 (AREVA, 2014).

### 3.7.1 Method

The ice-induced flooding evaluation followed the HHA described in NUREG/CR-7046, Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America (NRC, 2011, Section 2).

With respect to ice effects, the HHA used the following two steps:

1. Review historical ice events and backwater effects due to ice jams in the Mississippi River in the vicinity of WSES.
2. Evaluate historical water temperatures to assess the possibility of the formation of ice jams in the Mississippi River in the vicinity of WSES.

### 3.7.2 Ice-Induced Flooding Results

#### 3.7.2.1 Review of historical ice events

The USACE maintains records of historical ice jams and dams on the Ice Jam Database (USACE, 2014a), which can be queried (using state name) to obtain information regarding historical ice events. There are no historic records of ice jams in the Mississippi River near WSES within the USACE Ice Jam Database.

#### 3.7.2.2 Review of Water Temperatures in the Vicinity of WSES

Water temperature data for the Mississippi River is available for USGS gage in Baton Rouge, Louisiana for the period 2007 – 2013 (USGS, 2014). The Baton Rouge gage is about 100 river miles upstream of WSES. The Mississippi River water temperature data indicate that water temperatures in the Mississippi River were always above freezing during the period of record. The annual minimum water temperatures recorded during the period ranges from 36°F to 48°F.

For the period 2000 – 2013, water temperature data for the Mississippi River at Natchez, Mississippi were obtained from the USACE, Vicksburg District (USACE, 2014b). The Natchez gage is located approximately 230 river miles upstream of WSES. The data indicates that temperatures in the Mississippi River at Natchez during this period range from 33°F to 91°F.

### 3.7.3 Conclusions

At WSES, the potential of ice-induced flooding impacting the site is judged to be negligible for the following reasons.

Water temperature data from the USGS and USACE indicate that water temperatures in the Mississippi River are above freezing. The formation of frazil ice is unlikely because water temperatures below freezing are required for a sustained period of time for the development of frazil ice. Frazil ice, ice jams, and ice dams are therefore not expected to form in the Mississippi River in the area of WSES. This conclusion is supported by information contained in the USACE Ice Jam Database, which indicates that no ice jams have been recorded in the Mississippi River near WSES.

In addition, the Lower Mississippi River (including the Mississippi River segment is heavily navigated, and USACE New Orleans District maintains navigable conditions (USACE, 2014c). This active management of the river further reduces the potential for ice jams. Therefore, ice-induced flooding at WSES due to ice effects is not anticipated.

### 3.7.4 References

**AREVA, 2014**. AREVA Document No. 32-9227011-000, "Waterford Steam Electric Station Flooding Hazard Re-evaluation - Ice Induced Flooding", 2014.

**NRC, 2011**. "NUREG/CR-7046: Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America", U.S. Nuclear Regulatory Commission, Springfield, VA, National Technical Information Service, 2011.

**USACE, 2014a**. "Ice Jam Database", U.S. Army Corps of Engineers, Ice Engineering Research Group, Cold Regions Research and Engineering Laboratory, https://rsgisias.crrel.usace.army.mil/apex/f?p=273:1:, Date accessed June 27, 2014, Date updated May 12, 2014.

**USACE, 2014b**. "Discharge Measurements", US Army Corps of Engineers, Vicksburg District, http://155.76.244.230/offices/ed/edh/Natchez_Flows.mht, Date accessed July 8, 2014, Date updated June 27, 2014.

**USACE, 2014c**. "Navigation", US Army Corps of Engineers, New Orleans District, Date accessed July 9, 2014, Date updated November, 2013, http://www.mvn.usace.army.mil/Missions/Navigation.aspx.

**USGS, 2014**. United States Geologic Survey, Surface Water, Instantaneous Data, Date accessed July 9, 2014, Date updated July 9, 2014.

## 3.8 Channel Migration or Diversion

Natural channels may migrate or divert either away from or toward the site. The relevant event for flooding is diversion of water towards the site. There are no well-established predictive models for channel diversions. Therefore, it is not possible to postulate a probable maximum channel diversion event. Instead, historical records and hydro-geomorphological data should be used to determine whether an adjacent channel, stream, or river has exhibited the tendency to meander towards the site. (NRC, 2011, Section 3.8)

This section summarizes the Channel Migration or Diversion evaluation performed in AREVA Document No. 51-9227014-000 (AREVA, 2014).

### 3.8.1 Method

The channel migration and diversion flooding evaluation followed the HHA approach described in NUREG/CR-7046, Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America (NRC, 2011).

With respect to channel migration and diversion, the HHA used the following two steps:

1. Review historical records and hydro-geomorphological data to assess whether the Mississippi River has exhibited the tendency to meander towards the site.
2. Evaluate present-day channel protection and stabilization measures in place to mitigate channel diversion of the Mississippi River.

### 3.8.2 Results

The Mississippi River has a highly sinuous course and has historically demonstrated morphological changes as attested to by the presence of abandoned channel courses. However, in the region of the river adjacent to the site, major morphological changes are not prevalent. Unlike at most of the Mississippi River channel segments upstream of WSES, neck and chute cutoffs are not present at the site (Saucier, 1994). The lack of abandoned channels can be attributed to the tough Pleistocene clays soils which make up the natural levees and are more resistant to erosion (Fisk, 1944). Bank caving is considered to be at a minimum between Donaldsonville and New Orleans, and WSES is located between these two cities (Fisk, 1944).

The USACE maintains navigability and operates flood control works on the Mississippi River which provide additional stability and flood flow relief to the Mississippi River. The Lower Mississippi River, including the Mississippi River segment which borders WSES, is frequently navigated. The USACE New Orleans District is responsible for channel improvements, dredging, and navigation maintenance activities on the Lower Mississippi River in the vicinity of WSES. As part of this mission, USACE New Orleans maintains over 360 miles of concrete mats and trenchfill revetments including the Waterford Revetment, which extends along the levee adjacent to the WSES site (USACE, 2014). Revetments are used to maintain and stabilize the channel alignment and are constructed using specialized dredges, towboats, survey boats, and other river-related equipment. The Waterford Revetment was constructed along the western bank of the river in the vicinity of WSES by the USACE. Based on the 1961, 1973, and 1991 Hydrographic Surveys, the Waterford Revetment was completed between 1973 and 1991 (USACE, 1961; USACE, 1973; USACE, 1991). The current Waterford Revetment is in place from river mile 125.5 to river mile 129.8 (USACE, 2011).

There are also four major floodways and diversion structures that may affect flow near WSES: the Old River control structure complex (located at River Mile 315), the West Atchafalaya Floodway (located at River Mile 302), the Morganza Floodway (located at River Mile 285), and the Bonnet Carre Spillway and Floodway (located at River Mile 128). The Old River control structures were completed in 1962 and modified in 1986 to prevent the migration of the Mississippi River into the alignment of the Atchafalaya River (USACE, 2009). Failure or mis-operation of these structures could result in significant portions of the flow of the Mississippi River being routed away from the current channel and into the Atchafalaya River. However, this would not result in increased risk of flooding or erosion to WSES because the Atchafalaya River is located more than 50 miles from the site. The floodplain area between the Atchafalaya River and the site will provide significant flow attenuation such that water surface elevations are not expected to threaten SSCs important to safety at WSES.

The Morganza, West Atchafalaya, and Bonnet Carre floodways only divert flow from the Mississippi River during periods of flooding (USACE, 2007). These engineered diversions would decrease the risk of channel migration at WSES due to flooding or erosion.

### 3.8.3 Conclusions

The Lower Mississippi River (including the Mississippi River segment which borders WSES) is frequently navigated, and the USACE New Orleans District is responsible for maintaining navigable conditions. As part of this responsibility, USACE actively maintains revetments and flood control structures that have been constructed to minimize the risk of channel diversions, bank erosion, and instability. The absence of major morphological changes in the region of the river adjacent to the site indicates that the river channel segment bordering WSES has not migrated in the past even though other parts of the river have exhibited a tendency to migrate. Furthermore, the bank of the river which borders WSES (which is an outer bank) is stabilized with revetments and levees, both of which are maintained by the USACE. Channel diversion therefore, does not present a credible risk as a flood mechanism at WSES.

### 3.8.4 References

**AREVA, 2014**. AREVA Document No. 51-9227014-000, "Waterford Steam Electric Station Flooding Hazard Re-Evaluation – Channel Diversion Flooding," 2014.

**Fisk, 1944.** Fisk, Harold, "Geological Investigation of the Alluvial Valley of the Lower Mississippi River," Mississippi River Commission, U.S. Army Corps of Engineers, December 1, 1944.

**NRC, 2011**. "NUREG/CR-7046: Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America, " U.S. Nuclear Regulatory Commission, Springfield, VA, National Technical Information Service, 2011.

**Saucier, 1994**. Saucier, Roger T. "Geomorphology and Quaternary Geologic History of the Lower Mississippi Valley," Mississippi River Commission, U.S. Army Corps of Engineers, December 1994.

**USACE, 1961**. "Mississippi Hydrographic Survey Book 1961", US Army Corps of Engineers, Published 1961.

**USACE, 1973**. "Mississippi Hydrographic Survey Book 1973", US Army Corps of Engineers, Published 1973.

**USACE, 1991**. "Mississippi Hydrographic Survey Book 1991", US Army Corps of Engineers, Published 1991.

**USACE, 2007**. "The Mississippi River and Tributaries Project: Floodways", Information Paper, Mississippi River Commission, U.S. Army Corps of Engineers, 2007.

**USACE, 2009**. "Old River Control", Brochure, U.S. Army Corps of Engineers New Orleans District, January, 2009.

**USACE, 2011**. "Mississippi River Channel Improvement Master Plan", US Army Corps of Engineers, Mississippi Valley Division, March 2011.

**USACE, 2014**. "Channel Improvement and Stabilization Program", U.S. Army Corps of Engineers, New Orleans District, http://www.mvn.usace.army.mil/Missions/Engineering/ChannelImprovementandStabilizationProgram.aspx, Date accessed: July 13, 2014, Date updated July 2, 2014.

## 3.9 Combined Effect Flood

This section addresses combined events flooding at WSES in accordance with guidance presented in NUREG/CR-7046 – Design Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States (NRC, 2011). This section summarizes the evaluation of combined flooding events performed in the AREVA Calculation 32-9227036-000, "Waterford Steam Electric Station Flooding Hazard Re-Evaluation – Combined Effects" (AREVA, 2015d).

### 3.9.1 Methodology

The criteria for assessing combined effect floods are provided in NUREG/CR-7046, Appendix H (NRC, 2011). Three of the five scenarios presented NUREG/CR-7046 apply to WSES: H.1 – Floods caused by precipitation events, H.2 – Floods caused by seismic dam failures, and H.3 – Floods along shores of open and semi-enclosed bodies of water. The other two combined effect flood scenarios described in NUREG/CR-7046 (i.e., H.4 – Floods along shores of enclosed bodies of water and H.5 – Floods caused by tsunamis) were screened out as not being applicable to WSES.

H.1 - Floods caused by precipitation events

The criteria include the following:

- Alternative 1 - A combination of mean monthly base flow, median soil moisture, antecedent or subsequent rain, the Probable Maximum Precipitation (PMP), and waves induced by 2-year wind speed applied along the critical direction;
- Alternative 2 - A combination of mean monthly base flow, probable maximum snowpack, a 100-year, snow-season rainfall, and waves induces by 2-year wind speed applied along the critical direction; and
- Alternative 3 - A combination of mean monthly base flow, a 100-year snowpack, snow-season PMP, and waves induced by 2-year wind speed applied along the critical direction.

H.2 - Floods caused by seismic dam failures

The criteria include the following:

- Alternative 1 – A combination of a 25-year flood, a flood caused by dam failure resulting from a safe shutdown earthquake (SSE) and coincident with the peak of the 25-year flood, and waves induced by 2-year wind speed applied along the critical direction;
- Alternative 2 – A combination of the lesser of one-half of PMF or the 500-year flood, a flood caused by dam failure resulting from an operating basis earthquake (OBE) and coincident with the peak of one-half of PMF or the 500-year flood, and waves induced by 2-year wind speed applied along the critical direction.

H.3 - Floods along the Shores of Open and Semi-Enclosed Bodies of Water

The criteria include the following:

- Alternative 1 – The combination of the lesser of one-half of the Probable Maximum Flood (PMF) or the 500-year flood, the surge and seiche from the worst regional hurricane or windstorm with wind-wave activity, and the antecedent 10 percent exceedance high tide;
- Alternative 2 – The combination of the PMF, the 25-year storm surge and seiche with wind-wave activity, and the antecedent 10 percent exceedance high tide;
- Alternative 3 – The combination of the 25-year flood in the stream, the Probable Maximum Storm Surge (PMSS) and seiche with wind-wave activity, and the antecedent 10 percent exceedance high tide; and

- Alternative 4 – For drainage areas less than 300 square miles in hurricane-prone areas, a combination of the PMF, the Probable Maximum Hurricane (PMH) in the open or semi-enclosed water body, and the antecedent 10 percent exceedance high tide.

Alternative 4 does not apply because the drainage area of the Mississippi River at WSES is 1.2 million square miles (and exceeds the 300 square mile drainage area criterion) (AREVA, 2014b).

Due to the location of WSES, on the west bank of the Mississippi River next to the Mississippi River levees, failure of the levees was considered under Scenarios H.1, H.2, and H.3. The Mississippi River levee near WSES was designed for a maximum water surface elevation of 25.1 feet, MSL. The difference between the design maximum water surface elevation and the top of levee crest elevation of 30 feet, MSL (referred to as freeboard) is 4.9 feet (WSES, 2012). As discussed in the NRC ISG on dam failures (NRC, 2013a), levees are typically not designed to withstand high water levels for long periods and should be assumed to fail when their design elevations are exceeded for long periods of time, or when overtopped, unless it can be demonstrated through detailed engineering analysis, supported by site-specific information, that the levee can withstand such loading conditions. Levee failure was simulated using the two-dimensional model, FLO-2D (AREVA, 2014b).

The combined effect scenarios included an assessment of wave effects at WSES (i.e. at the NPIS). The siting of the NPIS is such that wave effects at the south, west and north sides of NPIS will be insignificant due to physical obstructions to wave formation and propagation (Figure 3-66). However, the east side of the NPIS is less developed with physical obstructions to wave formation and propagation. The wave heights and standing wave crest elevations were calculated and compared to the elevation of the east side of the NPIS wall to determine whether overtopping could occur. The standing wave crest elevation was calculated using the Sainflou formula (Sainflou, 1928). The wave overtopping flowrate (if the standing wave crest elevation exceeded the top elevation of the NPIS) was calculated using the Franco formula (USACE, 2006).

The flooding impacts of Scenarios H.1, H.2 and H.3 combined effect flood mechanisms were assessed and are summarized below.

#### 3.9.1.1 H.1 - Floods caused by precipitation events

The bounding alternative under this scenario is a combination of the PMF plus upstream dam failure (AREVA, 2014c) and waves induced by 2-year wind speed, applied along the critical direction, since snow/ice accumulation is not significant at WSES. Under H.1, the water level in the Mississippi River will be higher than the levee design elevation across the levee network. Failure of the levee may therefore occur anywhere along the levee network. Based on the HHA approach, the breach location was conservatively (relative to other distant breach locations) considered to be somewhere across from the site.

The 2-year annual recurrence interval wind speed was calculated using the Generalized Extreme Value (GEV) Distribution (MathWorks, 2015) and recorded wind speed data from the Louis Armstrong New Orleans International Airport, LA gage (Global Surface Summary of Day Data (GSOD) WBAN ID = 12916). This station is located about 13 miles east of WSES. The 2-year annual recurrence interval wind speed was used to calculate the wave heights at the NPIS to determine if any overtopping into the NPIS would occur using the methodology presented in the USACE Coastal Engineering Manual (CEM) for wave growth with fetch (USACE, 2008). Due to the limited open area around the NPIS, fetch-limited conditions were considered appropriate for the calculation of wave effects at the WSES NPIS.

The computed flood depths and velocities were used to develop the hydrostatic force, as well as the hydrodynamic and debris loads against the NPIS wall.

#### 3.9.1.2 H.2 - Floods caused by seismic dam failures

Seismic dam failure with levee overtopping is, by inspection, bounded from a water surface elevation standpoint by H.1 above because both H.1 and H.2 include the same conservative assumption that all dams in the Lower

Mississippi River watershed fail (AREVA, 2014c). Therefore, the only difference between Scenarios H.1 and H.2 is that H.1 assumes a larger coincident flood (i.e., the PMF).

However, levee failure was conservatively analyzed under this scenario for the condition where the water level in the Mississippi River was assumed to be equal to the top elevation of the levee, without overtopping, and the vicinity of WSES was assumed to be dry. This condition was expected to be bounding in terms of maximum flood wave velocity near WSES since there was no backwater prior to the breaching of the levee that would reduce the hydraulic head for breach outflow or create other submergence effects at the levee breach location.

Using the HHA approach, a conservative methodology based on the American Society of Civil Engineers (ASCE) 7-10 guidance (ASCE, 2010), the standing wave crest elevation at the east side of the NPIS was calculated, to determine if there would be wave overtopping into the NPIS. ASCE guidance assumes a depth-limited wave condition, such that the standing wave crest reaches a height above the stillwater elevation of 1.2 times the depth at the wall (ASCE, 2010). This methodology results in a more conservative estimate of the standing wave crest elevation compared to calculating the wave height for the 2-year annual recurrence interval wind speed, and is consistent with NRC guidance in JLD-ISG-2012-06 (NRC, 2013b). The computed flood depths and velocities were used to estimate hydrostatic forces, as well as the hydrodynamic forces and debris loads against the NPIS wall.

### 3.9.1.3 H.3 - Floods along the Shores of Open and Semi-Enclosed Bodies of Water

The bounding alternative for this scenario was determined to be a combination of the 25-year flood in the Mississippi River, the PMSS including antecedent water level (AREVA, 2015b), and coincident wind-generated waves (see Section 3.9.3.3 for details of the screening of the other H.3 alternatives). This scenario was simulated using the ADCIRC+SWAN model (AREVA, 2015b). The 25-year flood flow and stage in the Mississippi River at WSES was calculated using two data sources : flow data at the USGS gage at Baton Rouge, LA and stage data from the USGS gage at Reserve, LA (located about nine miles upstream of WSES). The 25-year stage at Reserve was translated to the 25-year stage at WSES using historical water surface elevation information for the Mississippi River. The 25-year flowrate input to the ADCIRC+SWAN model was adjusted iteratively until a stage equal to the calculated 25-year stage was obtained. The 25-year flow and stage was estimated using USGS Bulletin 17B procedures (USDOI, 1982).

Synthetic storms representing the PMH meteorological parameters and landfall locations identified as resulting in the largest storm surge at the site in the PMSS calculation (AREVA, 2015b) were evaluated coincident with the 25-river flood and AWL. Using the HHA approach, the storm surge simulations were performed in a progressive, step-wise manner to assess the sensitivity of the resulting surge elevations at WSES to hurricane intensity decay. Two sets of simulations were performed:

a) PMH with post-landfall, onshore decay; and

b) PMH with offshore decay and post-landfall, onshore decay.

The methodology used in calculating the offshore decay rate is discussed in the Section 3.4.4.1.

Post-simulation adjustments to the model-predicted flood stillwater elevations were made to account for the difference in modeled AWL compared to the calculated AWL of 2.9 feet, NAVD88-2004.65 (AREVA, 2015b) and for projected 30-year ground subsidence at WSES of 0.3 feet (AREVA, 2015b).

An analysis of a coincident Mississippi River levee failure near WSES was then performed based on the predicted water levels, wave heights and directions in the Mississippi River from the H.3 controlling alternative. Hydraulic simulations of levee failure were performed using the FLO-2D model. The water level in the Mississippi River for the levee failure was assumed as a constant stage equal to the maximum water level in the Mississippi River. Coincident flood depths at WSES and the adjoining floodplain prior to levee failure were modeled as a constant stage using the maximum flood elevation results of the ADCIRC+SWAN simulation.

Wind-generated waves at WSES, including within the Mississippi River, were calculated by the third-generation wave model, SWAN, which is a component of the coupled ADCIRC+SWAN model. The coupled ADCIRC+SWAN model SL16 grid attributes restrict wind in areas where the surface feature roughness prevent significant wind stress development on the water surface. Consistent with the conditions surrounding WSES, the modeled wind-generated waves represent a fetch-limited condition. The SWAN calculated wave heights were used to calculate the maximum reflected wave height at the NPIS to determine whether overtopping will occur.

The computed flood depths and velocities from FLO-2D (including levee failure) were used to develop the hydrostatic force, as well as the hydrodynamic and debris loads against the NPIS wall.

### 3.9.2 Assumptions

The following justified assumptions were made as part of this re-evaluation:

1. Offshore and post-landfall decay was applied to the PMH parameters, based on the method adopted by the IPET project (Resio, 2007) and historical data (AREVA, 2015a).

2. Projected subsidence and sea level rise was calculated over a period of 30 years.

3. Projected subsidence rate for a 30-year period was assumed to be the same as the observed rate in the past.

4. For H.3, levee failure was initiated when the water level in the Mississippi River reaches its maximum. For H.1 and H.2, levee failure was initiated when the water level in the Mississippi River reached the top of levee elevation.

5. A breach in the levee near WSES was conservatively assumed to result in a minimal drop in water level in the Mississippi River based on historical water data which indicate that the duration of peak flooding in the Mississippi River is on the order of weeks due to the very large watershed of the Mississippi River at WSES. Based on the HHA approach, levee breach for Scenarios H.1, H.2 and H.3 was conservatively assumed to occur at a single location (e.g., multiple levee breaches would reduce the water level in the Mississippi River and thus reduce available hydraulic head to drive the levee failure flood). It is noted that the NRC ISG indicates that crediting intentional off-site levee failures will generally not be accepted by NRC.

    a. The levee failure assumed for the combined effect of Storm 402C PMSS occurs near the site because the specific storm track creates the highest wave heights in the Mississippi River channel near the site. Mississippi River levees downstream of the site are overtopped by the storm surge propogating from the GoM, raising the levels in the Mississippi River near the site.

6. A debris object weight of 2,000 pounds was assumed for the calculation of debris loads (ASCE, 2010).

7. Wave heights at WSES for H.1 and H.3 are assumed to be fetch-limited due to the presence of wooded wetlands in the vicinity of the site (see Figure 3-67) which will inhibit wave development. Wave heights at WSES for H.2 were conservatively assumed to be depth-limited.

8. The effects of the vehicle barriers were conservatively ignored in this calculation since they are not intended to serve as flood protection structures.

9. The calculated 25-year stage inherently assumes that the Bonnet Carre Spillway is open because the spillway was open during most of the largest historical floods used in calculating the 25-year stage.

10. The NPIS was assumed to be a reinforced concrete structure in the selection of the building structure coefficient for the calculation of the debris impact loads based on the high resolution orthoimagery of WSES (AREVA, 2014d).

11. No upstream screening was conservatively assumed for the selection of the blockage coefficient for the calculation of the debris impact loads.

12. Wave loads were not calculated against the NPIS because waves at the site were assumed to be fetch-limited, non-breaking waves based on the ADCIRC+SWAN model results and the landcover data (Figure 3-67).

13. Projected subsidence was not accounted for in elevation data in the ADCIRC+SWAN model mesh used in simulating Alternative 3 for floods along the shores of open and semi-enclosed bodies of water (Scenario H.3). An elevation adjustment equal to the calculated projected subsidence at WSES was applied to the calculated storm surge elevations.

14. Recently constructed (2010) Hurricane and Storm Damage Risk Reduction System (HSDRRS) structures/ modifications are not included in the ADCIRC+SWAN model mesh. The new HSDRRS is capable of defending against up to a 100 year level of storm surge (USACE, 2014).

15. The Bonnet Carre Spillway was modeled as closed in the SL16 ADCIRC mesh used for storm surge modeling.

16. Water levels in the Mississippi River were modeled in FLO-2D as a constant stage equal to the computed maximum stage in the Mississippi River for the controlling alternatives for floods along the shores of open and semi-enclosed bodies of water and floods caused by precipitation events (i.e. PMF plus upstream dam failure (AREVA, 2014c)) and equal to the top of levee elevation of 30 feet MSL (AREVA, 2014a) for floods caused by seismic events.

17. Initial water levels at the site, prior to the failure of the levees, were modeled in FLO-2D as a constant stage equal to either the PMF plus dam failure elevation at the site for floods caused by precipitation events (AREVA, 2014c) or the flood stillwater elevation at the site of resulting from the controlling alternative for floods along the shores of open and semi-enclosed bodies of water (Scenario H.3).

### 3.9.3 Results

The following sections describe the results of the evaluation of the combined effect flood at WSES.

#### 3.9.3.1 H.1 – Floods Caused by Precipitation Events

<u>Stillwater Elevation Calculation</u>

The Probable Maximum Stillwater Elevation resulting from the PMF with coincident upstream dam failure is 20.5 feet, MSL at WSES and 29.9 feet, MSL in the Mississippi River (AREVA, 2014c). The water level of 29.9 feet, MSL in the Mississippi River exceeds the Mississippi River levee PDF design elevation of 25.1 feet, MSL at WSES (WSES, 2012). The levee is therefore assumed to fail based on the guidance in the NRC ISG (NRC, 2013a). The levee failure location is conservatively assumed to be in the vicinity of the WSES based on the HHA approach. The failure initiation location was based on sensitivity analysis and evaluation of levee top elevations and cross section dimensions described in the Combined Effects calculation (AREVA, 2015d). The selected failure initiation location was the location at which the levee top elevation was relatively low and the levee cross-section was thinnest in the vicinity of WSES.

The computational domain of the FLO-2D model encompasses the WSES site and an approximately one mile long section of the nearby floodplain along the Mississippi River north of WSES. WSES is located at localized high ground. Flow outside the selected model domain is therefore not likely to influence flood depths at WSES (AREVA, 2014b). A FLO-2D model grid size was 20 feet by 20 feet. Grid elements that were completely within the aerial extent of a building were assigned elevations at least 5 feet higher than the elevations of the surrounding non-building elements (AREVA, 2014b). Selected grid elements along the perimeter of the NPIS for the purposes of reporting results at the site are 24,028 (Northwest corner of NPIS); 23,873 (Northeast side of NPIS); 23,224 (Eastern side of NPIS); 20,422 (Southeast side of NPIS); 23,865 (Northwest side of NPIS); and 14,692 (Southeast side of the ISFSI). Discretized time-stage relationships specified for the stage control elements along the Mississippi River end of the levees are shown in Table 3-39. The top elevation of the levee was set at an elevation of 30 feet, MSL in the FLO-2D model. The water levels in the Mississippi River were specified to start at 0 feet and increase to the maximum elevation in the Mississippi River for the scenario being analyzed within an hour. The discretized time-stage relationships specified for the stage control elements along the computational boundary of the model in the vicinity of WSES are shown in Table 3-27. The initial water levels at the site were specified to start at 0 ft and increase to the maximum initial water level at WSES for the scenario being analyzed within an hour.

The Vehicle Barrier System was not modeled in the FLO-2D model since it is not a flood protection structure (AREVA, 2015d). The Manning's roughness coefficient values for the grid elements generally range from 0.02 for concrete and asphalt surfaces to 0.20 for areas with short trees (AREVA, 2015d). The following levee breach parameters were used in the FLO-2D simulation (AREVA, 2015d):

a) Elevation of prescribed failure = 28.5 feet, NAVD88-2004.65;

b) Base elevation of levee failure = 16 feet, NAVD88-2004.65;

c) Maximum levee breach width = 500 feet,

d) Horizontal rate of levee breach opening = 5000 feet per hour,

e) Vertical rate of levee breach opening = 125 feet per hour [(5000/500)x(28.5-16)],and

f) Duration after prescribed failure elevation is exceeded before levee fails = 1 hour.

The results of the FLO-2D levee breach analysis are summarized in Table 3-29 and Figure 3-68 through Figure 3-71. The results indicate maximum flood stillwater elevations around the NPIS of approximately 23.4 feet, MSL on the northern side, approximately 22.8 feet, MSL on the eastern side, and approximately 23.2 feet, MSL on the western side. Maximum flow depths around the NPIS are approximately 6.3 feet on the northern side, approximately 5.5 feet on the eastern side, and 6.2 feet on the western side and maximum velocities around the NPIS are approximately 2.6 fps on the northern side, 3.1 fps on the eastern side and approximately 2.4 fps on the western side.

Wind Wave Activity

The 2-year annual recurrence interval wind speed was calculated to be 33 knots. The effective fetch length was calculated to be 2.4 miles (Figure 3-67). The fetch length is the maximum unobstructed path for wave generation. The presence of wooded wetlands and to a lesser extent, buildings, in the vicinity of the NPIS will inhibit wave development. The significant wave height and period were calculated to be 1.9 feet and 2.3 seconds, respectively. The maximum wave height was calculated to be 3.1 feet. These results are shown in Table 3-30. The maximum reflected/standing wave crest elevation based on the Sainflou equation (Sainflou, 1928) at WSES resulting from floods caused by precipitation events is calculated to be 27.7 feet, MSL. The calculated maximum wave crest elevation is less than the top elevation of the east side of the NPIS. No overtopping is expecting from Scenario H.1. A summary of the results is provided in Table 3-31.

Hydrostatic, Hydrodynamic and Debris Impact Loads

The hydrostatic loading calculations for the H.1 controlling alternative are presented in Table 3-32. The hydrostatic loading results range from a minimum of approximately 1,100 pounds per foot acting at elevation 19.1 feet, MSL at the northwestern side of the NPIS to a maximum of approximately 1,200 pounds per foot acting at elevation 19.2 feet, MSL at the northwestern corner of the NPIS.

The hydrodynamic loading calculations for the H.1 controlling alternative are presented in Table 3-33. The hydrodynamic loading results range from a minimum of 0 pounds per foot at the northeastern and southeastern sides of the NPIS to a maximum of approximately 70 pounds per foot acting at elevation 20.1 feet, MSL at the eastern side of the NPIS.

The debris impact calculations for the H.1 controlling alternative are presented in Table 3-34. The debris impact results range from a minimum of approximately 2,100 pounds at the southeastern side of the NPIS to a maximum of approximately 4,900 pounds at the eastern side of the NPIS.

### 3.9.3.2 H.2 – Floods Caused by Seismic Dam Failure

Stillwater Elevation Calculation The results of the FLO-2D levee breach analysis for the condition where the water level in the Mississippi River is equal to the top elevation of the levees without overtopping and the site is dry are shown in Table 3-29 and Figure 3-72 through Figure 3-75. FLO-2D model development and levee failure parameters were as described in Section 3.9.3.1. The results indicate maximum stillwater elevations around the NPIS of approximately 20.8 feet, MSL on the northern side, approximately 18.9 feet, MSL on the eastern side, and approximately 20.7 feet, MSL on the western side. Maximum flow depths around the NPIS are approximately 3.7 feet on the northern side, approximately 1.6 feet on the eastern side, and approximately 3.7 feet on the western side and maximum velocities around the NPIS are approximately 2.1 fps on the northern side, approximately 3.6 fps on the eastern side and approximately 4.8 fps on the western side.

Wind Wave Activity

Waves were conservatively analyzed as depth-limited (i.e., 0.78 times maximum depth of water). The depth-limited wave at the eastern end of the NPIS is calculated as 1.2 feet.

The standing wave height at the east side of the NPIS is calculated as 1.9 feet (i.e., 1.2 times maximum depth of water) based on ASCE 7-10 guidance (ASCE, 2010).

The maximum standing wave crest elevation at WSES resulting from floods caused by precipitation events was calculated to be 20.8 feet, MSL at the east side of the NPIS. A summary of the results in feet, MSL is provided in Table 3-31.

Hydrostatic, Hydrodynamic and Debris Impact Loads

The hydrostatic loading calculations for the controlling alternative for floods caused by seismic dam failures are presented in Table 3-32. The hydrostatic loading results range from a minimum of approximately 10 pounds per foot acting at elevation 17.2 feet, MSL at the southeastern side of the NPIS to a maximum of approximately 440 pounds per foot acting at elevation 18.3 feet, MSL at the northwestern corner of the NPIS.

The hydrodynamic loading calculations for the controlling alternative for floods caused by seismic dam failures are presented in Table 3-33. The hydrodynamic loading results range from a minimum of 0 pounds per foot at the southeastern side of the NPIS to a maximum of approximately 120 pounds per foot acting at elevation 18.9 feet, MSL at the northwestern side of the NPIS.

The debris impact calculations for the controlling alternative for floods caused by seismic dam failures are presented in Table 3-34. The debris impact results range from a minimum of approximately 0 pounds at the southeastern side of the NPIS to a maximum of approximately 5,800 pounds at the northwestern side of the NPIS.

### 3.9.3.3 H.3 – Floods along the shores of open and semi-enclosed bodies of water – Streamside Location

Alternative 1

Alternative 1 was screened-out based on engineering judgment as follows:

1. Historical, observed stream gage data indicate that that extreme river floods (such as the one-half PMF or 500-year flood) in the Mississippi River occur during the months from January to July of each year (AREVA, 2015d).
2. The 500-year flood (which is the less than one-half of the PMF) was evaluated including levee failure in the WSES combined effect calculation (AREVA, 2015d). Flood depths at WSES resulting from a levee failure under flood conditions comparable to the 500-year in the Mississippi River do not result in the controlling flood depths at WSES.
3. Hurricane Katrina which was the "worst regional hurricane" and created the largest coastal storm surge within the GoM did not result in flooding at WSES. Sensitivity runs performed based on two selected historical events, Hurricane Camille 1969 and Hurricane Katrina 2005 did not result in flooding at WSES (AREVA, 2015d).
4. The parameters of a "worst regional hurricane" that would coincide with a 500 year flood were further evaluated. The Atlantic hurricane season officially runs from June 1 to November 30 each year, according to NOAA (Tropical Cyclone Climatology, NOAA, 2014). The seasonal trend indicates that the hurricane frequency peaks during the months of August through October (NOAA, 2014; AREVA, 2015a). As noted above, extreme river floods (such as the one-half PMF or 500-year flood) in the Mississippi River occur during the months from January to July of each year. The maximum coastal wind speed recorded in June/July is approximately 120 kt and occurred during Hurricane Dennis 2005 (AREVA, 2015d). Dennis made landfall near Pensacola as a Category 3 hurricane with a wind speed of 105 to 110 kt, a forward speed of 13 kt and a radius of maximum winds of 10 nm. NOAA's tropical cyclone report (NOAA, 2005) states "*Hurricane Dennis is an unusually strong July major hurricane ...*" The highest coastal surge caused by Hurricane Dennis 2005 was up to 7 feet above normal tide, near Santa Rosa Island (NOAA, 2005). The storm parameters representative of the months of June and July are not expected to result in a coastal surge significant enough to effect flooding at WSES.

Alternative 2

The resulting stage at WSES for Alternative 2 (not including Mississippi River failure) is calculated to be 20.0 feet, MSL. This stage is less than the resulting stage at WSES from Alternative 3 and would be essentially equivalent to Scenario H.1 given that WSES is located 120 miles upstream of the Gulf of Mexico. Therefore, no further considerations were made for Alternative 2.

Alternative 3

The representative storms, including storm track and meteorological parameters, that were analyzed are summarized in Table 3-35. These synthetic storms represent the PMH meteorological parameters and landfall locations identified as resulting in the largest storm surge at the site in the PMSS calculation (AREVA, 2015b) and were simulated coincident with the 25-river flood and AWL.

a) Antecedent Water Level and the 25-Year Flood Flow/Stage Calculation

The 25-year flow in the Mississippi River at Baton Rouge and the Atchafalaya River at Krotz Springs, Louisiana were calculated to be 1,465,600 cfs and 561,500 cfs, respectively (Table 3-37). The 25-year stage in the Mississippi River at WSES was calculated to be 23.1 feet, NAVD88 2004.65 or 24.5 feet, MSL (Table 3-37).

A separate AWL simulation from the one done in the PMSS calculation (AREVA, 2015b) was done using the ADCIRC+SWAN model to include the 25-year flood in the Mississippi River. The river flow at the upstream boundary in ADCIRC was iteratively adjusted until the calculated stage at WSES was achieved in ADCIRC. The applied flow to achieve the calculated flood 25-year stage elevation at WSES in ADCIRC was approximately 50 percent higher than the calculated 25-year flow based on the gage data. The required increase in flow to achieve the 25-year flood stage is likely a function of simplified Mississippi River geometry in the SL16 mesh and inconsistencies between the SL16 representation of the top of levee elevation and actual top of levee elevations (e.g., the SL16 mesh occasionally under-predicts the top of levee elevation, allowing some flow to escape the Mississippi River system).

The simulated AWLs along the coastline for the combined effects flood calculation are shown in Table 3-36. The average simulated antecedent water level along the Gulf coast from Eugene Island to Dauphine Island (Table 3-29) is 2.3 feet, NAVD88-2004.65 (approximately 0.6-foot lower than the calculated AWL of 2.9 feet, NAVD88-2004.65 (AREVA, 2015b)). Note that the post-simulation adjustments for AWL along the coastline in the PMSS calculation and the combined effect calculation are different because of differences in initial ocean level elevations used in the two calculations. These differences are explained in detail in the PMSS calculation (AREVA, 2015b) and combined effect calculation (AREVA, 2015d).

b) PMSS with 25-year River Flood

Synthetic storms representing the PMH meteorological parameters and landfall locations identified as resulting in the largest storm surge at the site in the PMSS calculation (AREVA, 2015b) were re-evaluated coincident with the 25-river flood and AWL. Two general sets of simulations were performed using the coupled ADCIRC+SWAN models, the first set was PMH without offshore decay but with post-landfall, onshore decay; and the second set was PMH with both offshore and post landfall, onshore decay.

The peak stillwater levels from the ADCIRC+SWAN simulations of combined PMSS and 25-year river flood and AWL are presented in Table 3-35. The resulting unadjusted maximum water levels for each of the two sets of simulations are as follows:

i. 25.0 feet, MSL for the PMH with post-landfall, onshore decay simulations (Storm 302); and

ii. 23.1 feet, MSL for the PMH with both offshore and post-landfall, onshore decay simulations (Storm 402c).

Maximum wind speed plots across the Gulf of Mexico for the Storms 302 and 402c are shown in Figure 3-77 and Figure 3-78, respectively. Maximum wind speed plots near WSES for Storms 302 and 402c are shown in Figure Figure 3-79 and Figure 3-80, respectively.

Maximum unadjusted water level plots for the Storms 302 and 402c are shown in Figure 3-81 and Figure 3-82, respectively. Maximum unadjusted water level plots near WSES for Storms 302 and 402c are shown in Figure 3-83 and Figure 3-84, respectively.

Post-simulation adjustments to the model-predicted flood stillwater elevations were made including:

1) the simulated flood stillwater elevations at WSES linearly adjusted (increased) by 0.6 foot to account for the difference in the simulated and calculated AWL; and

2) the simulated flood stillwater elevations at WSES were linearly adjusted (increased) to account for estimated future (i.e., 30-years or 50-years) ground subsidence at WSES by adding 0.3 foot (AREVA, 2015b).

The total post simulation adjustment for the ADCIRC+SWAN results for the 400 series storms (Table 3-35) including Storm 402c are:

0.6 feet (AWL adjustment) + 0.3 feet (projected subsidence adjustment) = <u>0.9 foot</u>

Note that the AWL and subsidence adjustment for the 300 series storms including Storm 302 is based on the adjustment calculated in the WSES PMSS calculation (AREVA, 2015b) and is equal to <u>0.7 foot</u>.

The adjusted maximum water levels for the two sets of simulations are shown in Table 3-35 and summarized below as follows:

i. 25.7 feet, MSL for the PMH without offshore decay but with post landfall, onshore decay simulations (Storm 302); and

ii. 24.0 feet, MSL for the PMH with both offshore and post landfall, onshore decay simulations (Storm 402c).

c) Coincident Wave Activity in the Mississippi River and Mississippi River Levee Failure

The resulting ADCIRC+SWAN maximum significant wave crest elevations in the Mississippi River for Storm 302 and Storm 402c are 35.9 feet, MSL and 33.5 feet, MSL respectively (Table 3-38). The maximum wave crest elevations in the Mississippi River for both storms exceed the typical top of levee elevation of 30 feet, MSL in the vicinity of WSES. The wave results shown in Table 3-38 and Figure 3-85 indicate that wave actions during both Storm 302 and Storm 402c are such that wave overtopping of the right Mississippi River levee near WSES occurs. The track and parameters for Storm 302 and 402c were selected such that the worst wind and surge conditions overland and in the Mississippi River occur in the vicinity of the site (see Figure 3-81, Figure 3-82, Figure 3-83 and Figure 3-84). Failure of the right descending Mississippi River levee resulting from wave overtopping from Storms 302 and 402c is therefore more likely in the vicinity of the site that anywhere else along the levee system. Levee failure simulations using FLO-2D were performed for Storm 302 and Storm 402c.

The initiating breach location in the vicinity of WSES was selected based on sensitivity analysis described in Section 3.9.3.1. For the levee failure simulation, discretized time-stage relationships were specified for the stage control elements along the Mississippi River end of the levees are shown in Table 3-39. The water levels in the Mississippi River were specified to start at 0 feet and increase to the maximum elevation of 29.9 feet, NAVD88-2004.65 or 31.3 feet, MSL (for Storm 302) and 28.1 feet, NAVD88-2004.65 or 29.5 feet, MSL (for Storm 402c) in the Mississippi River within an hour, based on the calculated maximum elevation in the Mississippi River (PMSS + 25-year river flood). The discretized time-stage relationships specified for the stage control elements along the computational boundary of the model in the vicinity of WSES are shown in Table 3-40. The initial water levels at the site were specified to start at 0 ft and increase to 24.3 feet, NAVD88-2004.65 (for Storm 302) and 22.6 feet, NAVD88-2004.65 (for Storm 402c) within an hour. The elevations of 24.3 feet, NAVD88-2004.65 and 22.6 feet, NAVD88-2004.65 are the calculated elevation at WSES resulting from the PMSS combined with the 25-year flood in the Mississippi River.

For Storm 302, the results indicate maximum stillwater elevations around the NPIS of approximately 26.6 feet, NAVD88-2004.65 (28.0 feet, MSL) on the northern side, and approximately 26.1 feet, NAVD88-2004.65 (27.5 feet, MSL) on the eastern side. Maximum flow depths around the NPIS are approximately 10.9 feet on the northern side, and approximately 10.1 feet on the eastern side. Maximum velocities around the NPIS are approximately 3.6 fps on the northern side, and approximately 2.8 fps on the eastern side.

The results of the levee breach analysis under this scenario are shown in Table 3-29 and Figure 3-86 through Figure 3-89 for Storm 402c. For Storm 402c, the results indicate maximum stillwater elevations around the NPIS of approximately 24.5 feet, NAVD88-2004.65 (25.9 feet, MSL) on the northern side, and approximately 23.9 feet, NAVD88-2004.65 (25.3 feet, MSL) on the eastern side. Maximum flow depths around the NPIS are approximately 8.9 feet on the northern side, and approximately 8.0 feet on the eastern side. Maximum velocities around the NPIS are approximately 3.3 fps on the northern side, and approximately 2.3 fps on the eastern side.

Wind Wave Activity

The wind-generated wave heights were calculated using the ADCIRC+SWAN model and are presented in Table 3-41. The maximum significant wave height during Storm 302 reached 3.6 ft at the east side of the NPIS, while the maximum significant wave height for 402c at the east side of the NPIS was 2.7 feet. Inspection of the simulated wave heights as well as the model attributes and land cover indicate that the simulated wave heights represent a fetch-limited wave condition, with simulated significant wave heights less than the theoretical depth-limited wave heights.

The standing wave crest elevations for Storms 302 and 402c at the east side of the NPIS were calculated to be 36.2 feet, MSL and 31.8 feet, MSL respectively.

Because the wave crest elevations exceed the NPIS minimum protection height of 29.18 ft MSL, overtopping flowrates were calculated for the two storms. The overtopping rate during each storm was calculated at each time step of the wave output time series, taking into account the wave direction at each time step. Wave overtopping of the NPIS will not occur when the waves are propagating away from the NPIS. The calculated wave overtopping flowrates were then multiplied by the length of the eastern section of the NPIS being overtopped to estimate the overtopping flowrate in cubic feet per second. The length of the eastern section of the NPIS is 128 feet (WSES, 2011). The peak wave overtopping rate resulting from Storm 302 was calculated to be 8.2 cfs and the peak wave overtopping rate from Storm 402c was calculated to be less than 0.1 cfs.

The estimated duration of significant and maximum wave overtopping from Storm 302 are about 5.3 hours and 7.3 hours respectively (Figure 3-90). Flooding from Storm 402c is only expected from the maximum wave height. The duration of overtopping of maximum wave overtopping from Storm 402c is about 5.3 hours (Figure 3-91). A summary of the results in feet, MSL is provided in Table 3-31.

Wave Overtopping Potential Combined with Rainfall

An evaluation of the significance of the peak overtopping rate from Storm 402c of 0.1 cfs combined with rainfall was performed using the results of the NPIS LIP calculation (AREVA, 2015c). As noted in the NPIS LIP calculation, the peak inflow to DCT Basin B due to the LIP is 15.9 cfs during the first 5-minute burst of rainfall (AREVA, 2015c). The inflow rate during hours 2 through 6 of the LIP (after the front-loaded, 1-hr LIP) is a steady 0.59 cfs. The peak wave overtopping rate of 0.1 cfsdue to Storm 402c is therefore relatively low compared to the peak LIP inflow rate.

The time required to reach a threshold ponding depth within DCT Basin B, the area of the NPIS subject to overtopping due to wave action, was calculated using a mass-balance approach as 28.9 hours. During the NPIS LIP, the maximum depth is attained after 1 hour (AREVA, 2015c). Therefore, the wave overtopping rate is insignificant relative to the NPIS LIP. Additionally, it should be noted that the duration of wave overtopping of the NPIS was calculated to be approximately 5.3 hours and, therefore, the NPIS threshold depth would not be attained through wave overtopping alone.

Hydrostatic, Hydrodynamic and Debris Impact Loads

The hydrostatic loading calculations based on the results for Storm 402c for floods along open bodies of water are presented in Table 3-32. The hydrostatic loading results range from a minimum of approximately 1,800 pounds per foot at the southeastern side of the NPIS to a maximum of approximately 2,500 pounds per foot at the northwestern corner of the NPIS.

The hydrodynamic loading calculations based on the results for Storm 402c for floods along open bodies of water are presented in Table 3-33. The hydrodynamic loading results range from a minimum of 0 pounds per foot at the southeastern side of the NPIS to a maximum of approximately 110 pounds per foot at the northwestern corner of the NPIS.

The debris impact calculations based on the results for Storm 402c for floods along open bodies of water are presented in Table 3-34. The debris impact results range from a minimum of approximately 1,800 pounds at the southeastern side of the NPIS to a maximum of approximately 5,300 pounds at the northwestern corner of the NPIS.

The potential hydrodynamic loading and debris loading resulting from the combined effect flood event are bound by the missile load requirements of the NPIS exterior wall (WSES, 1997). The flow velocity of flood waters at the NPIS wall are relatively low due to the configuration of the combined effect flood, and as a result, the hydrodynamic loads and debris impact loads are also relatively low.

Historical Storms

The GoM, including the Louisiana shoreline, has experienced several intense hurricanes with large storm surges. A few of these storms resulted in significant inland propagation of the coastal flood as well as surge propagation up the Mississippi River. Examples include:

- Gustav 2008 made landfall as a Category 2 storm near Dulac, LA, southwest of WSES (NOAA, 2009). The storm had a core size (Rmax) of 25 nm around landfall (AREVA, 2015a). Due to the low-lying floodplain along its storm track, there was significant inland propagation of the coastal storm surge. The storm surge generated by Gustav advanced inland to Lake Verret, located about 80 miles (70 nm) from the landfall location on the coast (USGS, 2014). See Figure 3-76 for the locations of Dulac, LA and Lake Verret.
- Hurricane Camille 1969 and Hurricane Katrina 2005 cause a significant surge impact along the coast and that the coastal surges propagate inland.
- Hurricanes Isaac and Katrina resulted in storm surges up to twelve feet up the Mississippi River (USGS, 2015 and WSES, 2015).

As a means of comparing the results of the combined effects / PMSS flood, storm surge simulations were performed using the coupled ADCIRC+SWAN model for synthetic storms with storm track and meteorological characteristics that are representative of the most significant (i.e., worst recorded storm surge) historical storms (Hurricanes Katrina and Camille). The landfalls were modified to evaluate the potential effect that these historical storms would have on WSES. The results of the simulations indicated that the WSES site is not flooded by these simulations.

#### 3.9.3.4 Error and Uncertainty

Uncertainty is typically characterized as consisting of both aleatory variability and epistemic uncertainty. Aleatory variability includes the statistical uncertainty that represents the natural randomness of a process (e.g., storm track direction, landfall, etc.). Epistemic uncertainty is the scientific uncertainty in the model or the process, due to limited data and knowledge about the process or model input.

The flood hazard evaluation results presented in this report do not include adjustments to account for potential uncertainty. The controlling flood event was determined using a deterministic methodology, which was focused on determining the most severe event that could reasonably occur at the WSES site. Based on sensitivity analyses performed using relocation of severe historical storms, the controlling combined effect flood methodology is considered conservative without the application of an uncertainty factor.

### 3.9.4 Conclusions

Combined effect flooding was evaluated as per the guidance in Appendix H of NUREG/CR-7046. The Controlling Combined Effect Flood (CCEF) scenario is considered to be the result of H.3, Storm 402c. The results of this flood scenario is a stillwater level at the NPIS of 26.0 ft MSL, a significant wave crest elevation of 26.9 ft MSL, a maximum wave crest elevation of 31.8 ft MSL, and a resulting overtopping rate at DCT B of 0.1 cfs. The duration of maximum wave overtopping from Storm 402c is about 5.3 hours

### 3.9.5 References

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**Table 3-29: Results Summary for Controlling Alternatives**

| | Repre-sentative Grid Element | Ground Surface Elevation (ft, NAVD88-2004.65) | Max Water Surface Elevation (ft, NAVD88-2004.65) | | Maximum Flow Depth (ft) | | Maximum Velocity (fps) | | Time to Maximum Flow Depth (hours) | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | H.1 | H.2 | H.1 | H.2 | H.1 | H.2 | H.1 | H.2 |
| Northwest corner of NPIS | 24,028 | 15.66 | 21.95 | 19.40 | 6.29 | 3.74 | 2.62 | 2.13 | 0.41 | 2.70 |
| Northeast side of NPIS | 23,873 | 15.88 | 21.89 | 19.22 | 6.01 | 3.34 | 1.38 | 1.80 | 0.39 | 2.67 |
| Eastern side of NPIS | 23,224 | 15.91 | 21.43 | 17.49 | 5.52 | 1.58 | 3.09 | 3.59 | 2.74 | 1.29 |
| Southeast side of NPIS | 20,422 | 15.63 | 20.66 | 16.06 | 5.03 | 0.43 | 1.32 | 0.77 | 0.37 | 1.88 |
| Northwest side of NPIS | 23,865 | 15.68 | 21.84 | 19.35 | 6.16 | 3.67 | 2.43 | 4.84 | 1.75 | 0.75 |
| Southeast side of the Independent Spent Fuel Storage Installation (ISFSI) | 14,692 | 14.75 | 19.72 | 15.97 | 4.97 | 1.22 | 2.07 | 3.14 | 0.35 | 0.51 |
| | | | H.3 | | | | | | | |
| | | | No offshore decay; Post landfall decay starting at landfall (302) | Offshore decay; Post landfall decay starting 1 hour after landfall (402c) | No offshore decay; Post landfall decay starting at landfall (302) | Offshore decay; Post landfall decay starting 1 hour after landfall (402c) | No offshore decay; Post landfall decay starting at landfall (302) | Offshore decay; Post landfall decay starting 1 hour after landfall (402c) | No offshore decay; Post landfall decay starting at landfall (302) | Offshore decay; Post landfall decay starting 1 hour after landfall (402c) |
| Northwest corner of NPIS | 24,028 | 15.66 | 26.57 | 24.54 | 10.91 | 8.88 | 3.63 | 3.34 | 0.06 | 0.34 |
| Northeast side of NPIS | 23,873 | 15.88 | 26.54 | 24.45 | 10.66 | 8.57 | 2.45 | 1.65 | 0.06 | 0.33 |
| Eastern side of NPIS | 23,224 | 15.91 | 26.05 | 23.93 | 10.14 | 8.02 | 2.76 | 2.34 | 0.06 | 0.47 |

| | Repre-sentative Grid Element | Ground Surface Elevation | Max Water Surface Elevation | | Maximum Flow Depth | | Maximum Velocity | | Time to Maximum Flow Depth | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | ft, NAVD88 2004.65 | (ft, NAVD88-2004.65) | | (ft) | | (fps) | | (hours) | |
| Southeast side of NPIS | 20,422 | | 25.04 | 23.13 | 9.41 | 7.5 | 1.87 | 1.13 | 0.07 | 1.11 |
| Northwest side of NPIS | 23,865 | 15.68 | 26.5 | 24.52 | 10.82 | 8.84 | 3.12 | 2.77 | 0.06 | 0.75 |
| Southeast side of the Independent Spent Fuel Storage Installation (ISFSI) | 14,692 | 14.75 | 24.64 | 22.86 | 9.89 | 8.11 | 2.87 | 3.44 | 0.08 | 1.04 |

Notes:

1) To convert elevations from NAVD88-2004.65 to MSL, add 1.43 ft to the NAVD88-2004.65 elevations.
2) Run with breach width of 500 ft, breach rate of 5,000 ft per hour, and at breach location 200 ft west of location 2 (grid cell 27,884) and breaches in the "South" direction.
3) Maximum water surface elevation reported includes the effects of levee breach. See Table 3-40 for water levels without levee breach.

**Table 3-30: Wave Results for H.1**

| | | |
|---|---|---|
| Fetch | 3860 | Meters (see Figure 44) |
| 2yr wind speed | 17 | m/s |
| Equations governing wave growth with fetch | | |
| Cd | 0.0017 | drag coefficient |
| u*^2 | 0.49 | friction velocity (m/s) |
| Tp | 2.28 | peak period, seconds |
| Hmo | 0.57 | significant wave height, meters |
| Hmo | 1.88 | significant wave height, ft |
| Hmax | 3.14 | maximum wave height, ft |

| **Sainflou Formula** | | |
|---|---|---|
| Elevation at NPIS | 17.3 | ft, MSL |
| Flood Depth | 5.5 | ft |
| Wave Length (L) | 19.0 | ft |
| Vertical shift | 1.7 | ft |
| H.1 Stillwater Elevation | 22.8 | ft, MSL |
| Maximum Standing Wave Crest Elevation | 27.7 | ft, MSL |

## Table 3-31: Summary of Water Level and Wave Overtopping Results

| Flood Mechanism | Summary of Results (ft, MSL) | | | |
|---|---|---|---|---|
| **Floods caused by Precipitation Events (H.1)** | **Northwest NPIS** | | **East NPIS** | |
| PMF plus Upstream Dam Failure | 20.5 | | 20.5 | |
| PMF plus Upstream Dam Failure and Levee Failure | 23.4 | | 22.8 | |
| Significant Wave Height | N/A | | 1.9 | |
| Significant Wave Crest Elevation | N/A | | 23.8 | |
| Maximum Reflected/Standing Wave Crest Elevation | N/A | | 27.7 | |
| Wave Overtopping Flow (cfs) | N/A | | N/A | |
| **Floods caused by Seismic Events (H.2)** | **Northwest NPIS** | | **East NPIS** | |
| Levee Failure with water level at the top of levee elevation and with site dry | 20.8 | | 18.9 | |
| Depth Limited Wave Height | N/A | | 1.2 | |
| Depth Limited Wave Crest Elevation | N/A | | 19.6 | |
| Maximum Reflected/Standing Wave Crest Elevation | N/A | | 20.8 | |
| Wave Overtopping Flow (cfs) | N/A | | N/A | |
| **Floods Along the shores of open bodies of Water (H.3)** | **PMH without offshore decay (Storm 302)** | | **PMH with offshore decay (Storm 402c)** | |
| | **Northwest NPIS (Grid #24028)** | **East NPIS (Grid #23224)** | **Northwest NPIS (Grid #24028)** | **East NPIS (Grid #23224)** |
| PMSS plus 25-year River Flood Stillwater | 25.7 | 25.7 | 24.0 | 24.0 |
| PMSS plus 25-yr flood and Levee Failure Stillwater | 28.0 | 27.5 | 26.0 | 25.4 |
| Significant Wave Height | N/A | 3.6 | N/A | 2.7 |
| Significant Wave Crest Elevation | N/A | 29.5 | N/A | 26.9 |
| Maximum Reflected/Standing Wave Crest Elevation | N/A | 36.2 | N/A | 31.8 |
| Maximum Wave Overtopping Flow (cfs) | N/A | 8.2 | N/A | 0.1 |

Notes
1. Top elevation of lowest portion of the east side of the NPIS is 29.18 ft, MSL
2. The significant wave crest elevation was calculated as the sum of the stillwater elevation and fifty-five percent the significant wave height
3. Maximum waves are 1.67 times signficant wave heights. Significant waves represent the average of the highest 33 percent of waves. Maximum waves represent the highest one percent of waves.
4. Depth limited waves were therefore conservatively assumed for H.2

**Table 3-32: Hydrostatic Loading Calculation Results**

| Location | Representative Grid Element | Ground Surface Elevation (ft, NAVD88-2004.65) | H.1 | | H.2 | | H.3 | |
|---|---|---|---|---|---|---|---|---|
| | | | Loading (lb/ft) | Acting Elevation (ft, NAVD88-2004.65) | Loading (lb/ft) | Acting Elevation (ft, NAVD88-2004.65) | Loading (lb/ft) | Acting Elevation (ft, NAVD88-2004.65) |
| Northwest corner of NPIS | 24,028 | 15.66 | 1,234 | 17.8 | 436 | 16.9 | 2,460 | 18.6 |
| Northeast side of NPIS | 23,873 | 15.88 | 1,127 | 17.9 | 348 | 17.0 | 2,292 | 18.7 |
| Eastern side of NPIS | 23,224 | 15.91 | 951 | 17.8 | 78 | 16.4 | 2,007 | 18.6 |
| Southeast side of NPIS | 20,422 | 15.63 | 789 | 17.3 | 6 | 15.8 | 1,755 | 18.1 |
| Northwest side of NPIS | 23,865 | 15.68 | 1,184 | 17.7 | 420 | 16.9 | 2,438 | 18.6 |

Notes: (1) The scenario used for calculating Hydrostatic loading for H.3 is the ADCIRC+SWAN simulation with offshore decay and with post landfall decay starting one hour after landfall (402c). (2) To convert elevations from NAVD88-2004.65 to MSL, add 1.43 ft to the NAVD88-2004.65 elevations.

**Table 3-33: Hydrodynamic Loading Calculation Results**

| Location | Representative Grid Element | Ground Surface Elevation (ft, NAVD88-2004.65) | Obstruction Length (ft) (WSES, 2011) | H.1 | | H.2 | | H.3 | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | Loading (lb/ft) | Acting Elevation (ft, NAVD88-2004.65) | Loading (lb/ft) | Acting Elevation (ft, NAVD88-2004.65) | Loading (lb/ft) | Acting Elevation (ft, NAVD88-2004.65) |
| Northwest corner of NPIS | 24,028 | 15.66 | 47.75 | 39 | 18.8 | 23 | 17.5 | 111 | 20.1 |
| Northeast side of NPIS | 23,873 | 15.88 | 208.5 | 0 | N/A | 21 | 17.6 | 54 | 20.2 |
| Eastern side of NPIS | 23,224 | 15.91 | 169 | 69 | 18.7 | 39 | 16.7 | 50 | 19.9 |
| Southeast side of NPIS | 20,422 | 15.63 | 202.5 | 0 | N/A | 0 | N/A | 0 | N/A |
| Northwest side of NPIS | 23,865 | 15.68 | 100 | 38 | 18.8 | 115 | 17.5 | 55 | 20.1 |

Notes: (1) The scenario used for calculating Hydrostatic loading for H.3 is the ADCIRC+SWAN simulation with offshore decay and with post landfall decay starting one hour after landfall (402c). (2) To convert elevations from NAVD88-2004.65 to MSL, add 1.43 ft to the NAVD88-2004.65 elevations.

**Table 3-34: Debris Impact Calculation Results**

| Location | Representative Grid Element | Ground Surface Elevation (ft, NAVD88-2004.65) | H.1 | H.2 | H.3 |
|---|---|---|---|---|---|
| | | | Impact (lb) | Impact (lb) | Impact (lb) |
| Northwest corner of NPIS | 24,028 | 15.66 | 4,192 | 2,556 | 5,344 |
| Northeast side of NPIS | 23,873 | 15.88 | 2,208 | 1,440 | 2,640 |
| Eastern side of NPIS | 23,224 | 15.91 | 4,944 | 1,436 | 3,744 |
| Southeast side of NPIS | 20,422 | 2,112 | 2,112 | 0 | 1,808 |
| Northwest side of NPIS | 23,865 | 15.68 | 3,888 | 5,808 | 4,432 |

Notes: (1) The scenario used for calculating Hydrostatic loading for H.3 is the ADCIRC+SWAN simulation with offshore decay and with post landfall decay starting one hour after landfall (402c). (2) To convert elevations from NAVD88-2004.65 to MSL, add 1.43 ft to the NAVD88-2004.65 elevations.

**Table 3-35: ADCIRC+SWAN Results for Representative Storms (ft, NAVD88-2004.65)**

| STORM # | CPD (MB) | VMAX (KT) | BEARING (DEG FROM N) | FORWARD SPEED (KTS) | RMAX (NM) | LANDFALL POINT | DECAY | | WSES | | MISSISSIPPI RIVER NEAR WSES | INLAND DECAY START |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | OFFSHORE | INLAND | UNADJUSTED | ADJUSTED | | |
| 301 | 132 | 149 | -40 | 6 | 25 | 5 | NO | YES | 21.7 | **22.4** | 30.6 | at land fall |
| 401 | 132 | 149 | -40 | 6 | 25 | 5 | YES | YES | 17.7 | **18.6** | 30.5 | at land fall |
| 401a | 132 | 149 | -40 | 6 | 25 | 5 | YES | YES | 18.2 | **19.1** | 30.5 | 1-hr after land fall |
| | | | | | | | | | | | | |
| 302 | 132 | 149 | 0 | 6 | 25 | 3 | NO | YES | 23.6 | **24.3** | 29.9 | at land fall |
| 402 | 132 | 149 | 0 | 6 | 25 | 3 | YES | YES | 20.0 | **20.9** | 28.6 | at land fall |
| 402a | 132 | 149 | 0 | 6 | 25 | 3b | YES | YES | 21.2 | **22.1** | 27.9 | at land fall |
| 402b | 132 | 149 | 0 | 6 | 25 | 3 | YES | YES | 20.9 | **21.8** | 29.0 | 1-hr after land fall |
| 402c | 132 | 149 | 0 | 6 | 25 | 3b | YES | YES | 21.7 | **22.6** | 28.1 | 1-hr after land fall |
| | | | | | | | | | | | | |
| 303 | 132 | 149 | -60 | 6 | 25 | 9 | NO | YES | No Flooding | No Flooding | 29.6 | at land fall |
| 304 | 132 | 149 | -60 | 6 | 25 | 7 | NO | YES | 17.3 | **18.0** | 30.7 | at land fall |

Notes

1. 0.9 ft should be added to the ADCIRC computed elevations at to account for the difference in calculated Antecedent Water Level (AWL) and the simulated antecedent water level and for projected subsidence (except for 300 series runs). Adjustment for AWL and subsidence for 300 series runs is 0.7 ft (AREVA, 2015b)
2. Results are in ft, NAVD88-2004.65. mb = millibars; kt = knots; deg = degrees; nm = nautical miles. To convert elevations from NAVD88-2004.65 to MSL, add 1.43 ft to the NAVD88-2004.65 elevations.
3. Post landfall decay was applied to storms. Decay rate of -3 kt per hour and +2 mb per hour were applied for $V_{max}$ and central pressure ($P_o$), respectively.

**Table 3-36: Simulated Antecedent Water Levels along Coastline and in the Mississippi River**

| Location | Maximum Water Level (m, NAVD88-2004.65) | Maximum Water Level (ft, NAVD88-2004.65) |
|---|---|---|
| Eugene Island | 0.72 | 2.4 |
| Grand Isle | 0.70 | 2.3 |
| Dauphin Island | 0.70 | 2.3 |
| Average along Coastline | **0.71** | **2.3** |
| 25-year flood water level in Mississippi River (ft, NAVD88-2004.65) | | |
| **23.1** | | |

Note: To convert elevations from NAVD88-2004.65 ~~to~~ MSL, add 1.43 ~~ft~~ to the NAVD88-2004.65 elevations.

**Table 3-37: 25-Year and 500-Year Flood Flows and Stages**

| Gage | Extent of Years Analyzed | Number of Years with Data | 25-Year Flow (cfs) |
|---|---|---|---|
| Mississippi River at Baton Rouge, LA | 1932 - 2013 | 36 | 1,465,600 |
| Atchafalaya River at Krotz Springs, LA | 1935 - 1964 | 30 | 561,500 |

| Gage | Extent of Years Analyzed | Number of Years with Data | 25-Year Stage (ft, NAVD88 2004.65) | |
|---|---|---|---|---|
| | | | at Reserve, LA | At WSES |
| Mississippi River Stage at Reserve, LA | 1929 - 2013 | 85 | 25.0 | 23.1[1] |

| Gage | Extent of Years Analyzed | Number of Years with Data | 500-Year Flow (cfs) |
|---|---|---|---|
| Mississippi River at Tarbert Landing, MS | 1930 - 2014 | 85 | 1,825,000[2] |

Notes:

1. The 25-year stage at WSES was calculated using peak stage data at a gage nine miles upstream of WSES (in Reserve, LA) and translating that gage's 25-year stage to WSES based on historical water surface profiles for the Mississippi River. The 25-year stage at Reserve, LA was calculated as 25.0 ft NAVD88-2004.65. The calculated water surface profiles of the Mississippi River for the 2011 and 2008 floods indicated elevations at Reserve, LA to be 1.9 ft higher than elevations at WSES. Based on this relationship, when the elevation at Reserve, LA is 25.0 ft, the elevation at WSES is 23.1 ft.

2. This flow rate is 25,000 cfs greater than the calculated flow at WSES for the PMF.

**Table 3-38: Maximum Significant Wave Crest Elevations in Mississippi River near WSES**

| Storm # | Elevation in Mississippi River (ft, MSL) | Maximum Significant Wave Height in Mississippi River (ft) | Significant Wave Crest Elevation in Mississippi River (ft, MSL) | Wave Direction in Mississippi River near south levees (Deg.) |
|---|---|---|---|---|
| 302 | 31.3 | 8.3 | 35.9 | 190.1 |
| 402c | 29.5 | 7.3 | 33.5 | 190.1 |

Notes:
1. Geographical coordinate for wave output station in Mississippi River is (-90.470379 29.997851), which is in the north of WSES and adjacent to the levee.
2. Elevation of levee is about 30 ft, MSL.
3. Peak wave period and direction are at the same time of maximum significant wave height.
4. Wave direction is measured in Cartesian convention, counter clockwise from the positive x-axis indicating the direction waves are going toward.
5. Angle of levee is 156.4 degrees. Wave directions greater than 156.4 degrees imply waves overtopping levees.
6. The site vicinity was not flooded during Storm 303, so wave results are unavailable.

**Table 3-39: Time-Stage Relationship – Mississippi River Levee**

| Time (hours) | Stage (ft, NAVD88-2004.65) | | | |
|---|---|---|---|---|
| | Floods caused by precipitation events (H.1) | Floods caused by seismic dam failures (H.2) | Floods along shores of open and semi-enclosed bodies of water (H.3) | |
| | | | No offshore decay but with post landfall (302) | Offshore and post landfall decay (402c) |
| 0 | 0 | 0 | 0 | 0 |
| 1 | 28.6 | 28.6 | 29.9 | 28.1 |
| 10 | 28.6 | 28.6 | 29.9 | 28.1 |

Note: To convert elevations from NAVD88-2004.65 to MSL, add 1.43 ft to the NAVD88-2004.65 elevations.

**Table 3-40: Time-Stage Relationship – Initial Water Level at WSES**

| Time (hours) | Stage (ft, NAVD88-2004.65) | | | | |
|---|---|---|---|---|---|
| | Floods caused by precipitation events (H.1) | Floods caused by seismic dam failures (H.2) | Floods along shores of open and semi-enclosed bodies of water (H.3) | | |
| | | | No offshore decay but with post landfall (302) | | Offshore and post landfall decay (402c) |
| 0 | 0 | N/A | 0 | | 0 |
| 1 | 19.1 | N/A | 24.3 | | 22.6 |
| 10 | 19.1 | N/A | 24.3 | | 22.6 |

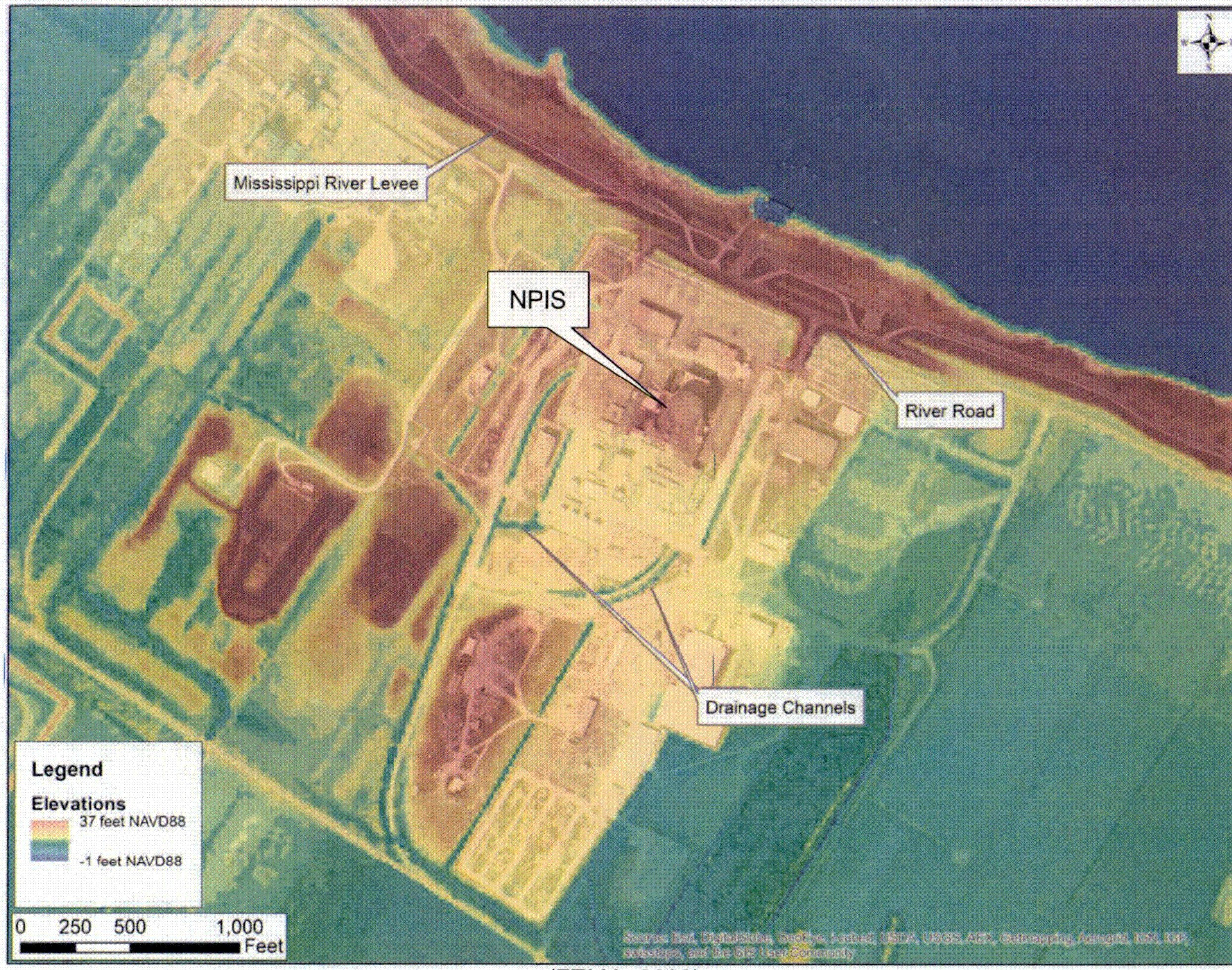
Note: To convert elevations from NAVD88-2004.65 to MSL, add 1.43 ft to the NAVD88-2004.65 elevations.

**Table 3-41: SWAN Outputs for Representative Storms at WSES**

| Storm # | Maximum Significant Wave height (ft) | Peak Wave Period (s) | Wave Direction (deg) |
|---|---|---|---|
| 301 | 2.8 | 3.2 | 92.4 |
| 401 | 1.2 | 2.7 | 94.6 |
| 401a | 1.4 | 3.0 | 92.8 |
| | | | |
| 302 | 3.6 | 3.2 | 72.4 |
| 402 | 2.1 | 2.7 | 68.6 |
| 402a | 2.5 | 2.7 | 68.7 |
| 402b | 2.4 | 3.0 | 66.2 |
| 402c | 2.7 | 3.0 | 67.6 |
| | | | |
| 303 | N/A | N/A | N/A |
| 304 | 1.1 | 1.8 | 212.8 |

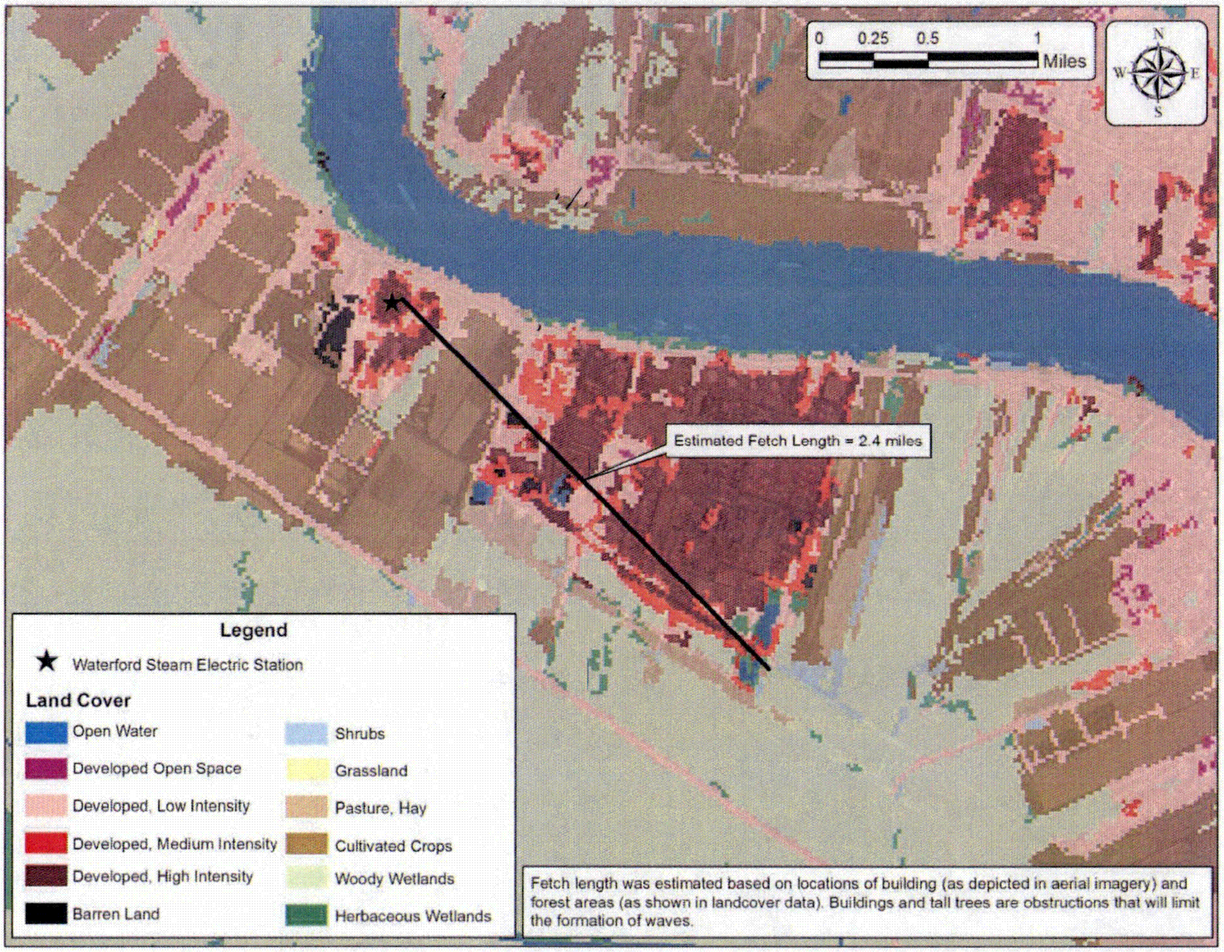
Notes:
1. Wave results are reported on Eastern Site of NPIS.
2. Wave period and direction are at the same time of maximum significant wave height.
3. Wave direction is measured in Cartesian convention, counter clockwise from the positive x-axis indicating the direction waves are going toward.
4. The site vicinity was not flooded during Storm 303, so wave results are unavailable.

**Figure 3-66: Topographic Relief of WSES**



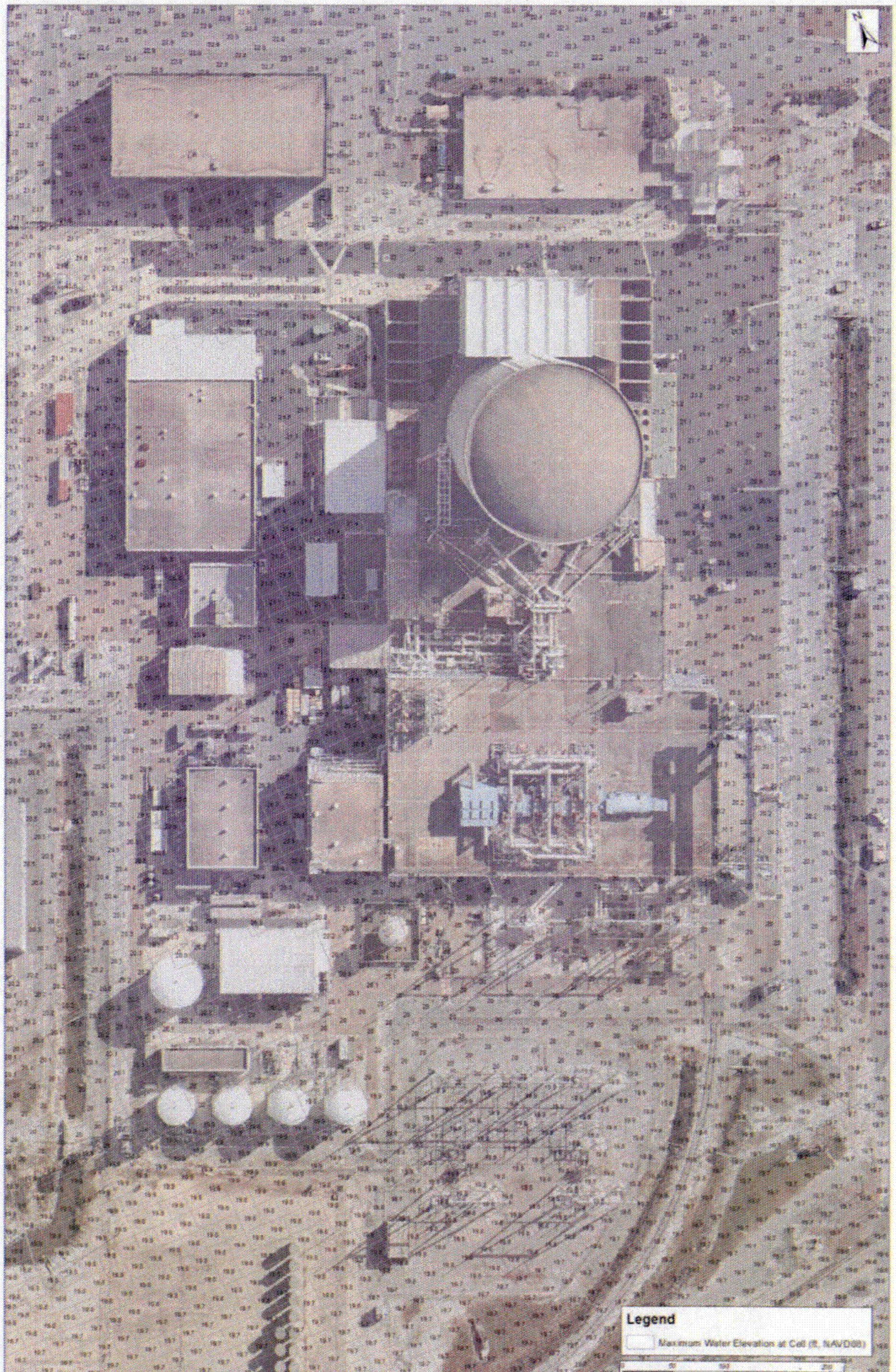

(FEMA, 2003)

**Figure 3-67: Effective Fetch Length (H.1)**




Any illegible text or features in this figure are not pertinent to the technical purposes of this document.

**Figure 3-68: FLO-2D - H.1 Maximum Water Surface Elevation near NPIS**




Elevations given in NAVD88-2004.65.
Any illegible text or features in this figure are not pertinent to the technical purposes of this document.

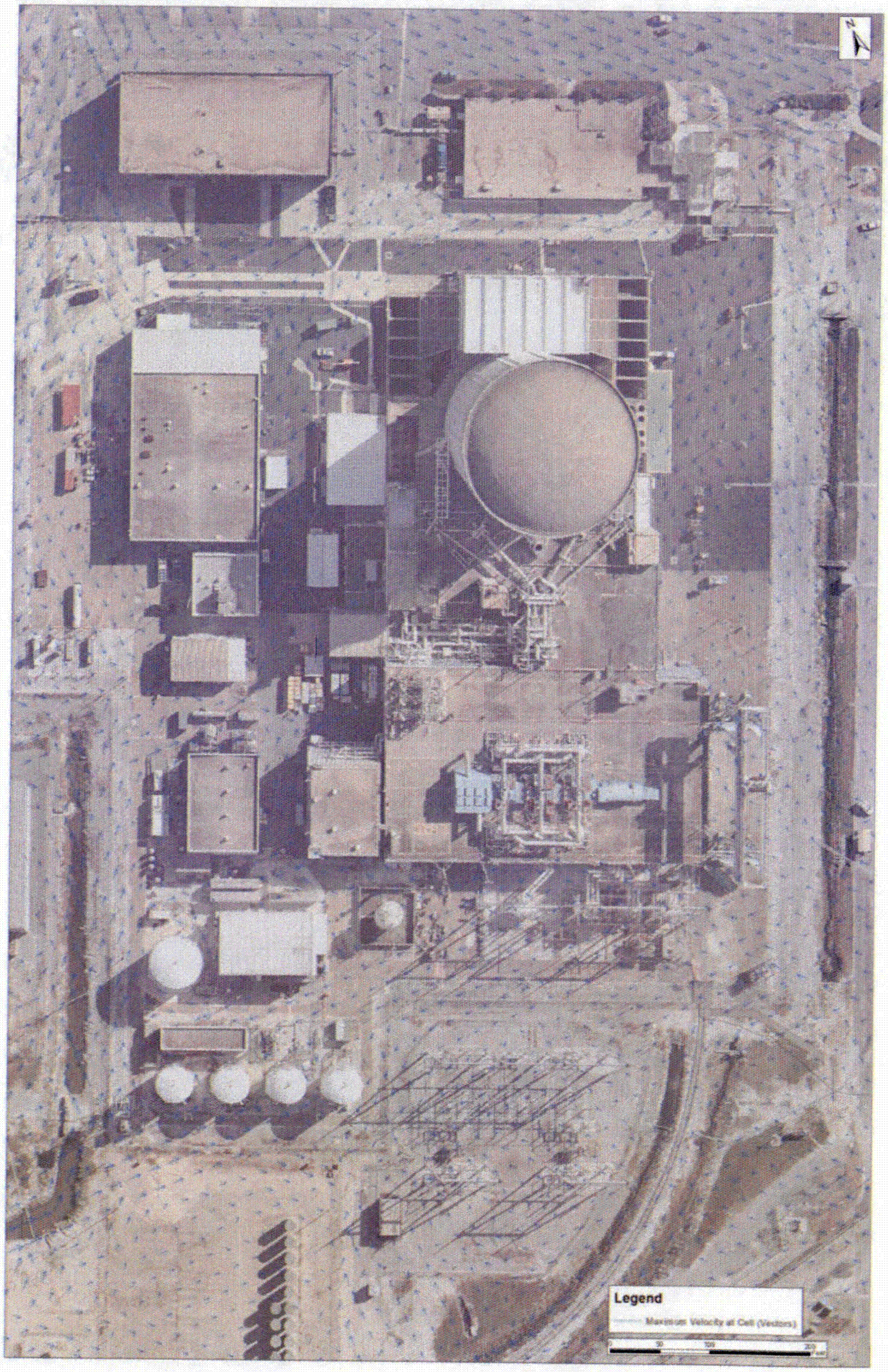
**Figure 3-69: FLO-2D - H.1 Maximum Flow Depth near NPIS**


Any illegible text or features in this figure are not pertinent to the technical purposes of this document.

Figure 3-70: FLO-2D - H.1 Maximum Velocity near NPIS

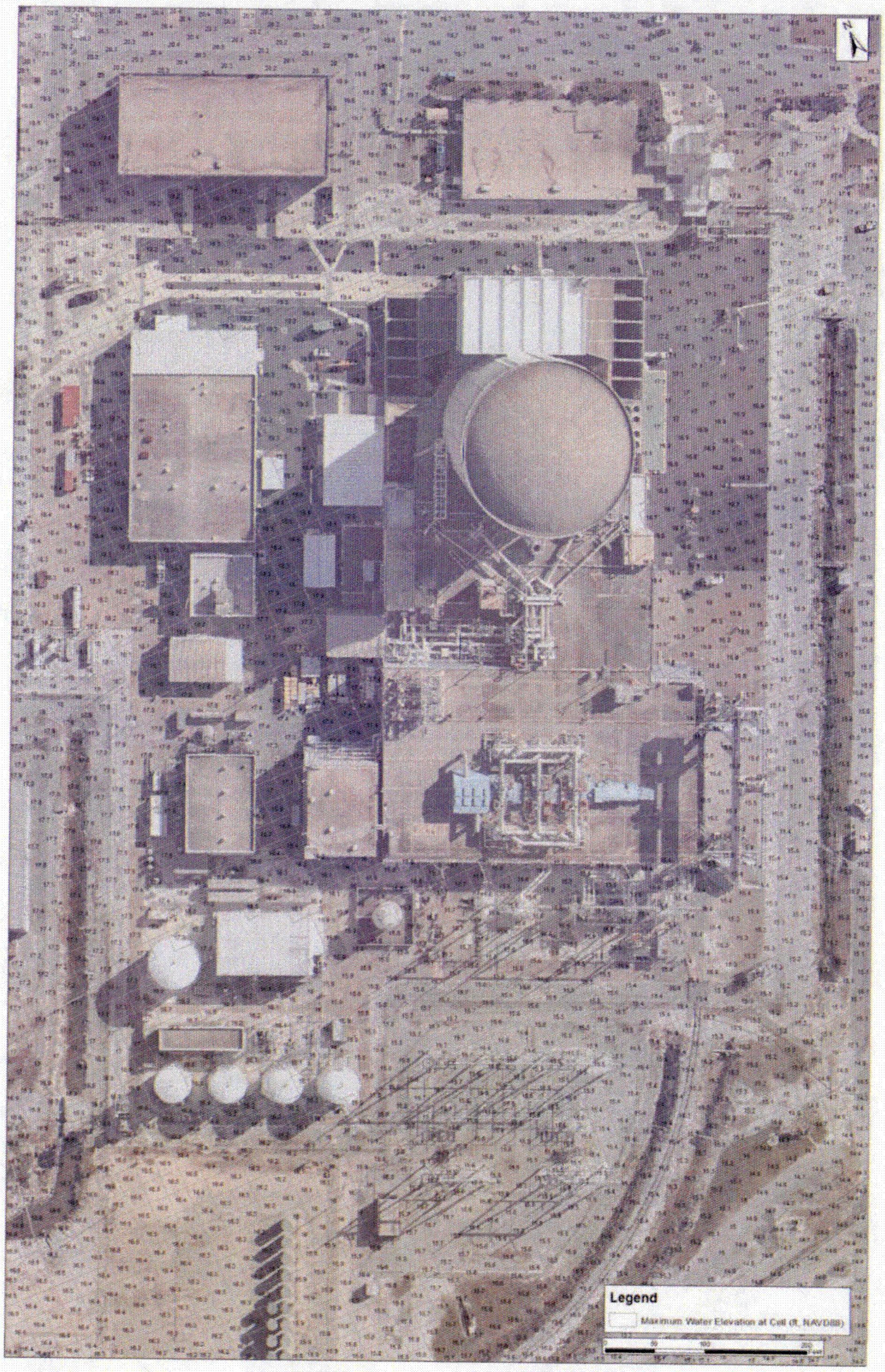

Any illegible text or features in this figure are not pertinent to the technical purposes of this document.

**Figure 3-71: FLO-2D - H.1 Maximum Velocity Vectors near NPIS**



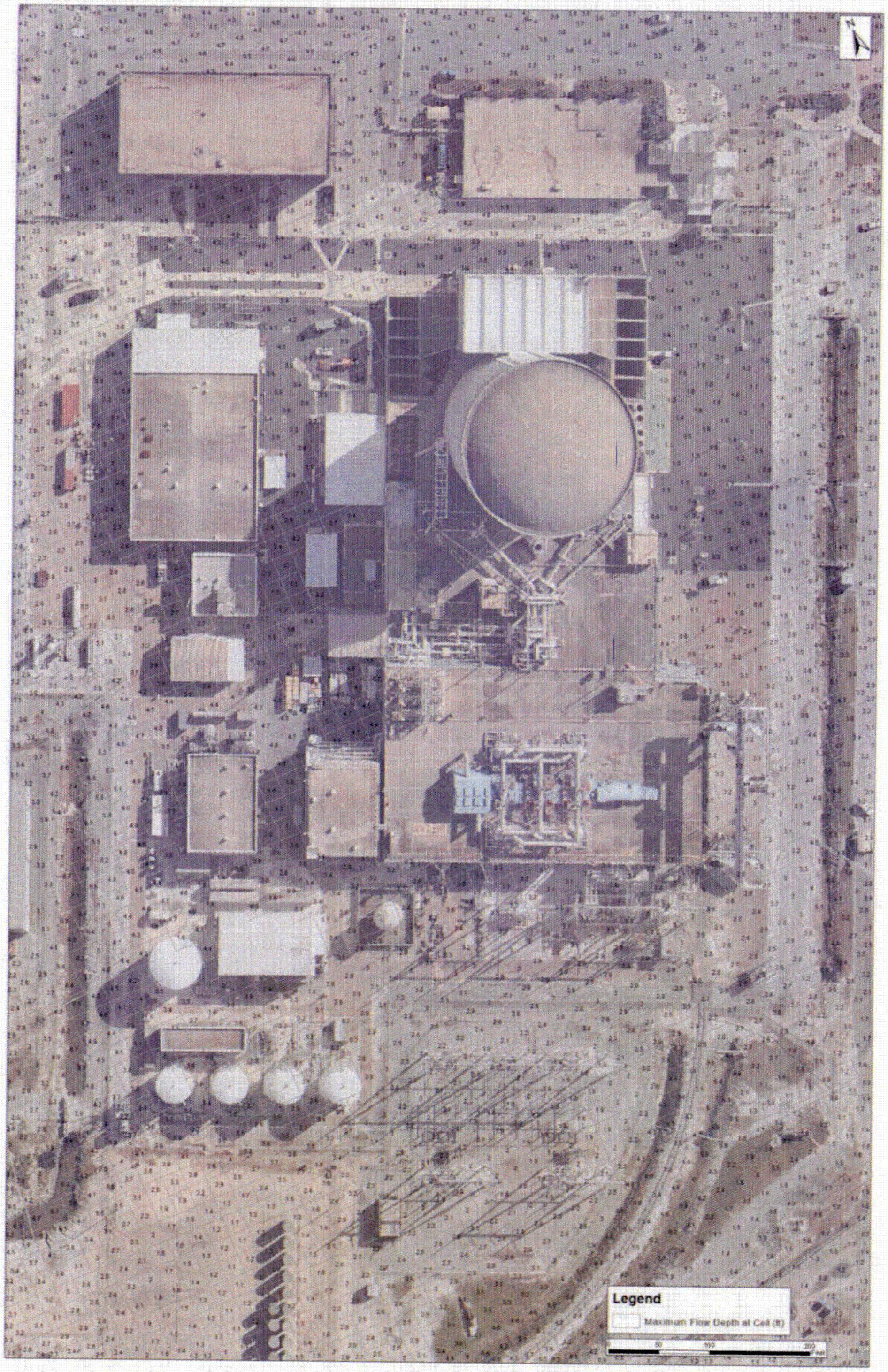

Any illegible text or features in this figure are not pertinent to the technical purposes of this document.

**Figure 3-72: FLO-2D - H.2 Maximum Water Surface Elevation near NPIS**




Elevations given in NAVD88-2004.65.
Any illegible text or features in this figure are not pertinent to the technical purposes of this document.

**Figure 3-73: FLO-2D - H.2 Maximum Flow Depth near NPIS**

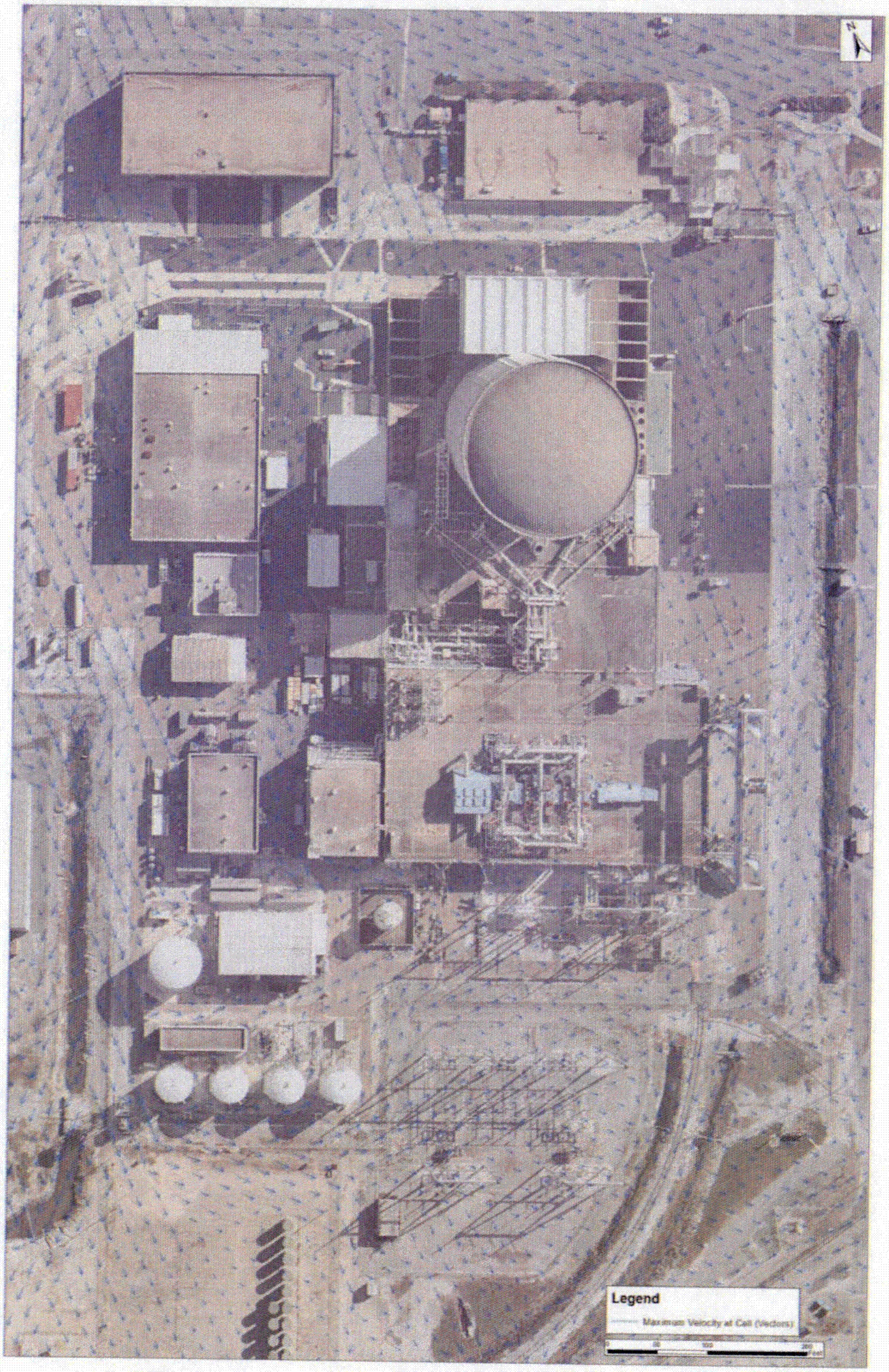



Any illegible text or features in this figure are not pertinent to the technical purposes of this document.

**Figure 3-74: FLO-2D - H.2 Maximum Velocity near NPIS**


Any illegible text or features in this figure are not pertinent to the technical purposes of this document.

**Figure 3-75: FLO-2D - H.2 Maximum Velocity Vectors near NPIS**




Any illegible text or features in this figure are not pertinent to the technical purposes of this document.

**Figure 3-76: Regional Aerial Photo Locus Map**

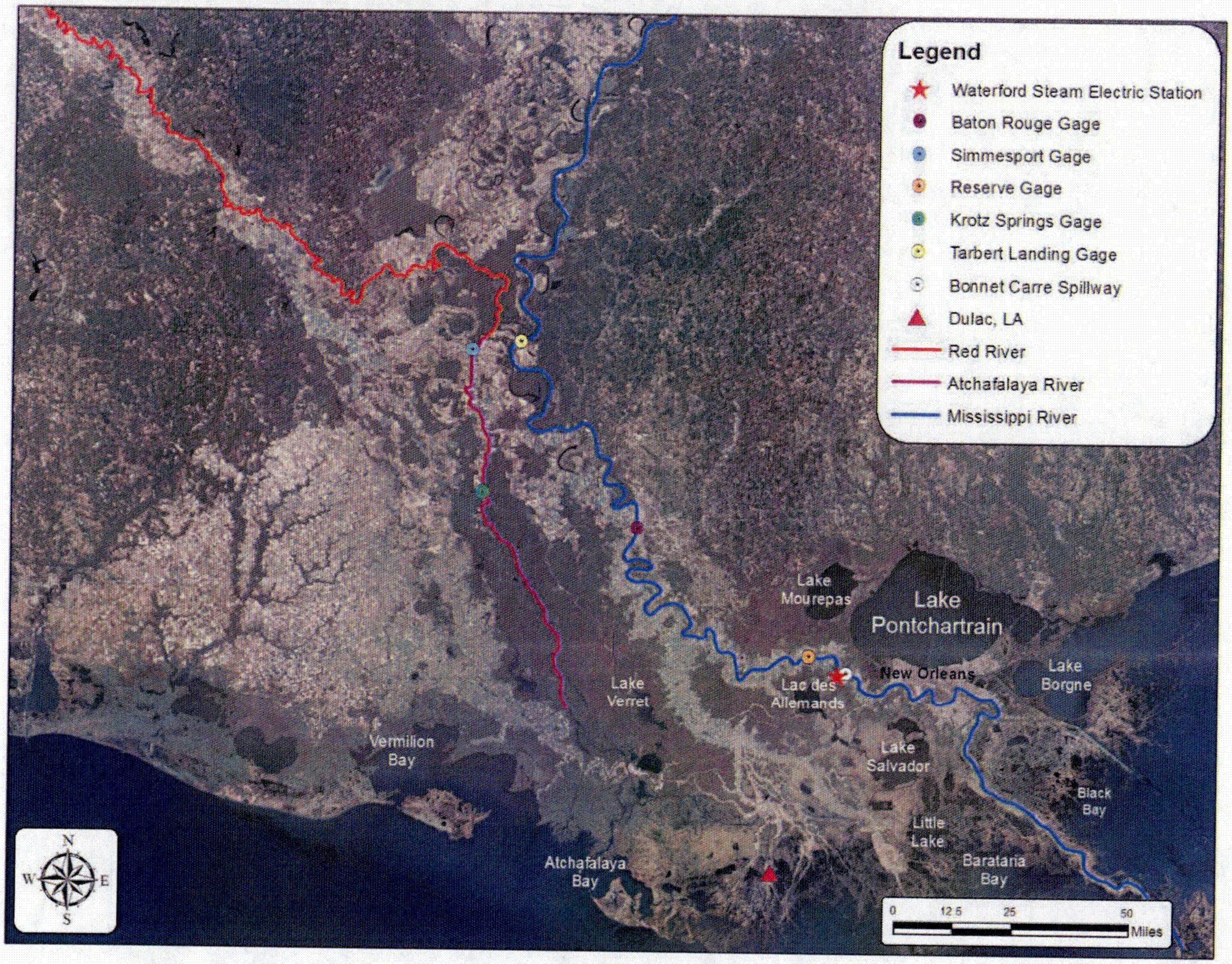
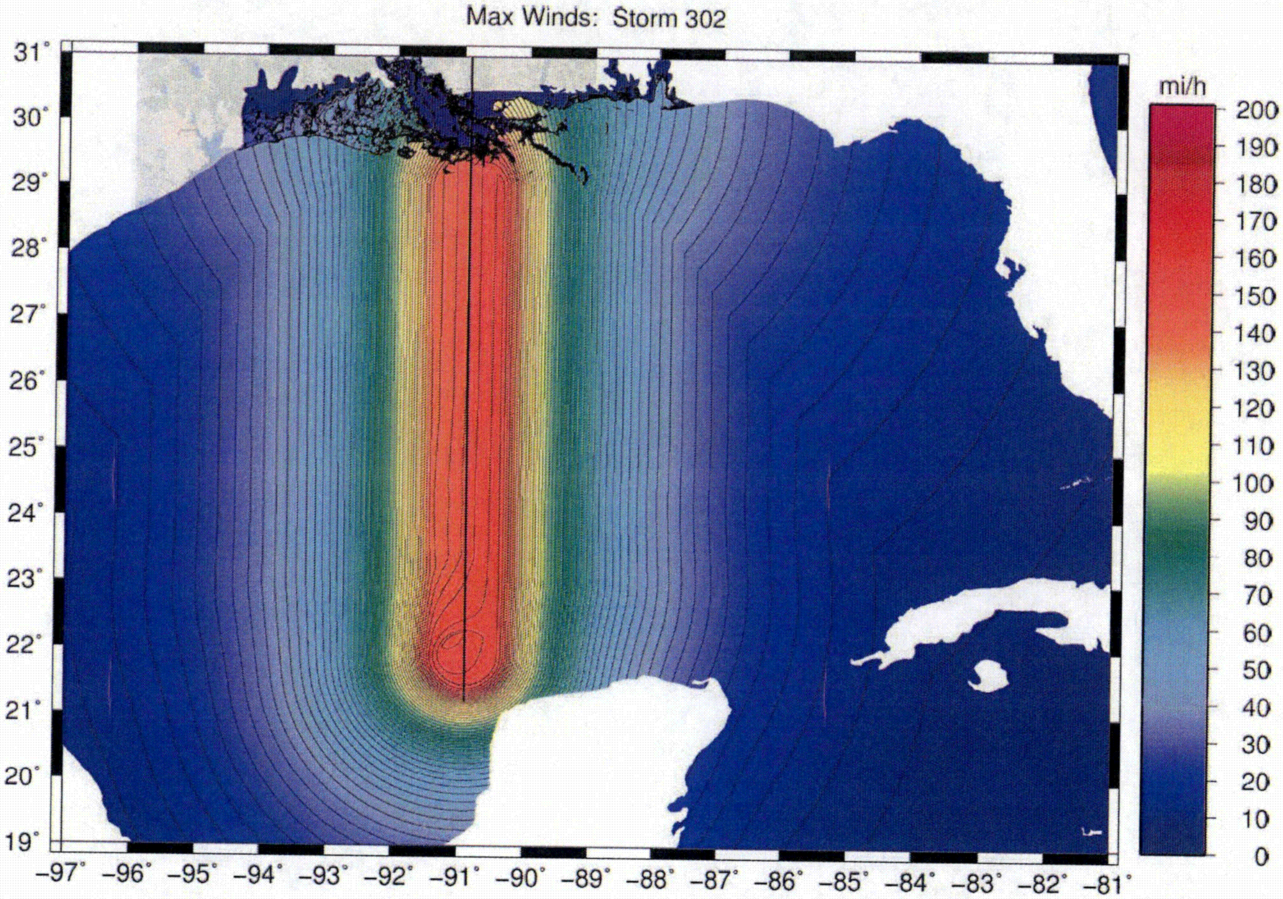
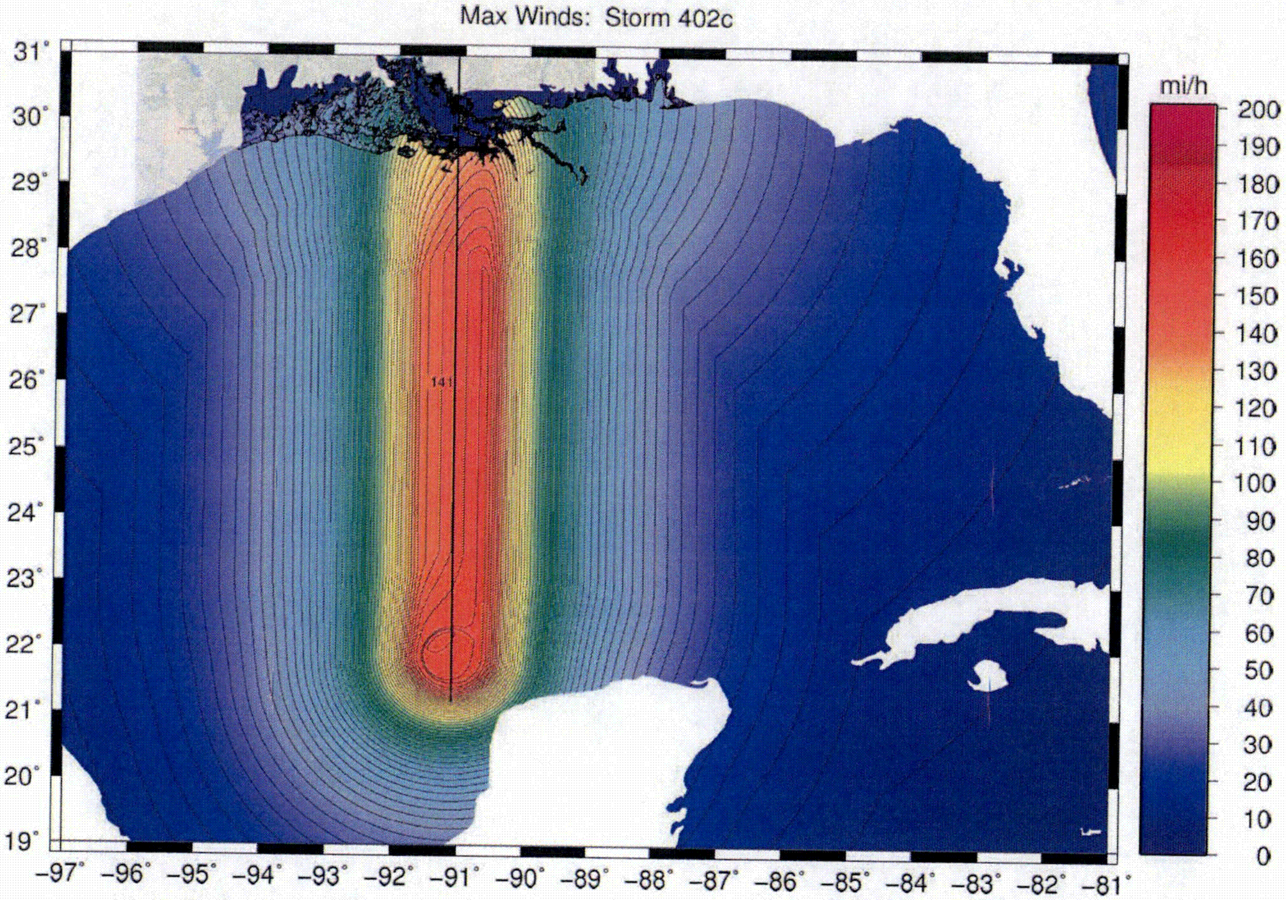


**Figure 3-77: Maximum 10-minute Winds in the Gulf of Mexico for Storm 302**



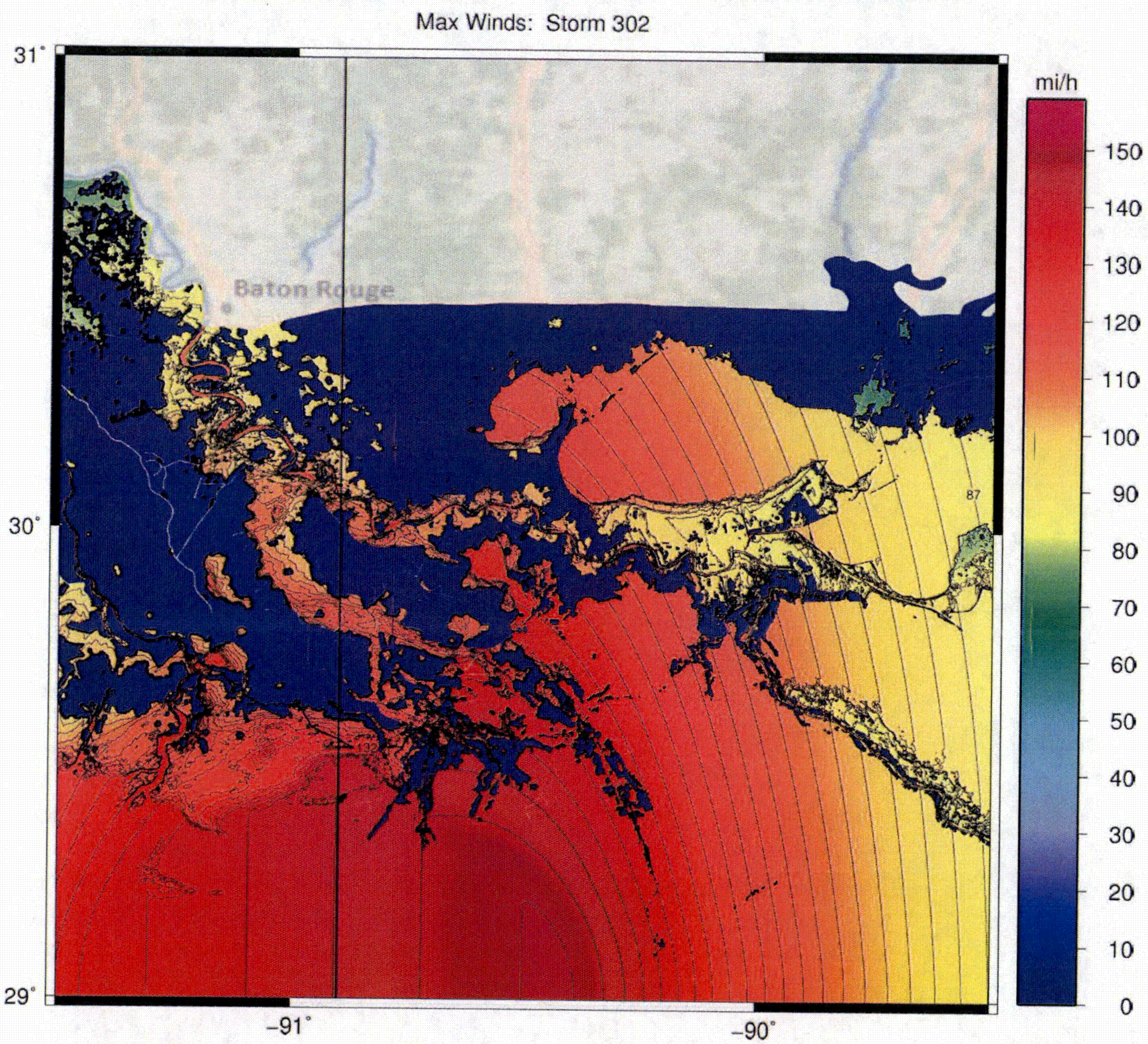

Note: Wind speed in miles per hour.

**Figure 3-78: Maximum 10-minute Winds in the Gulf of Mexico for Storm 402c**



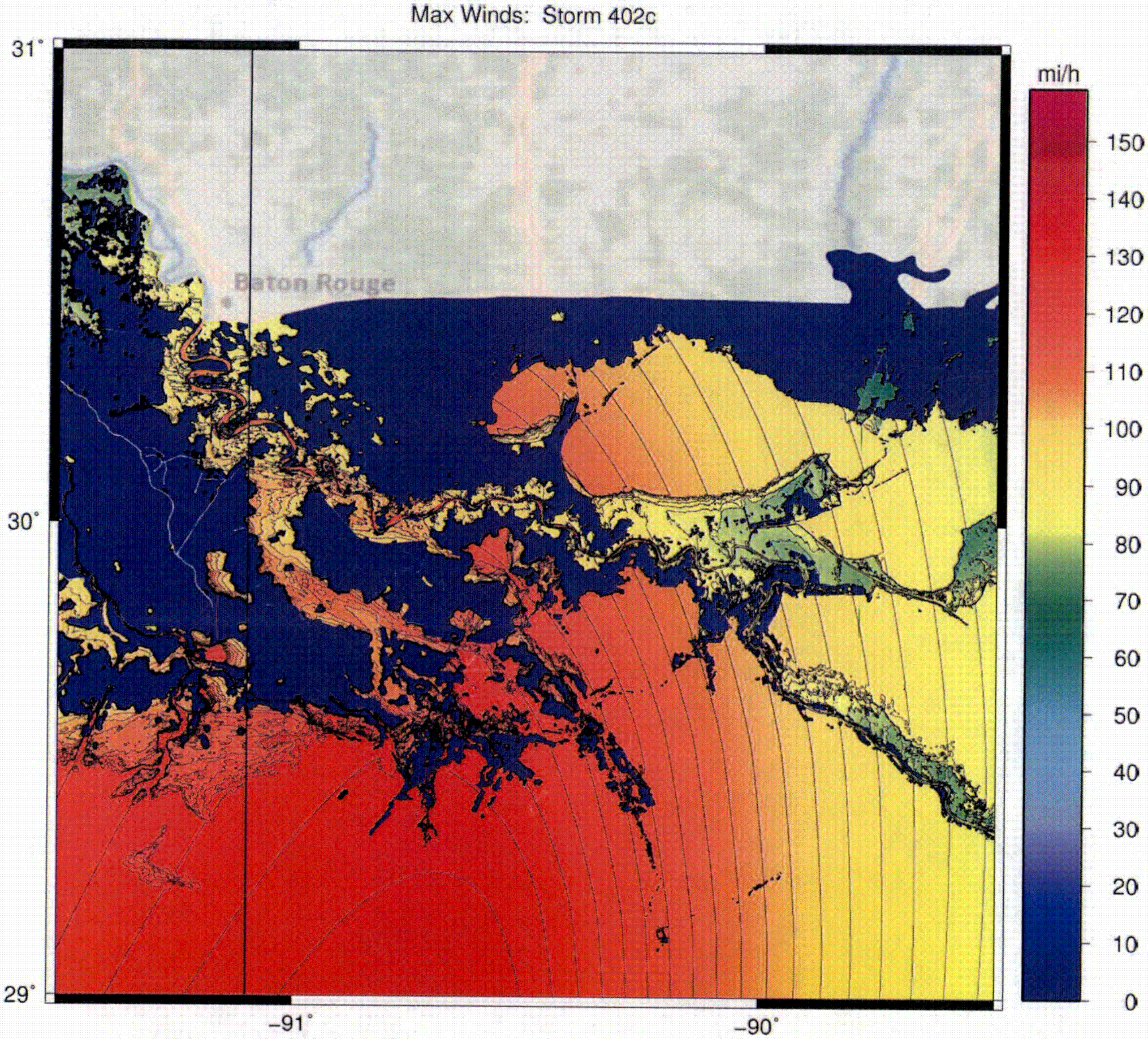

Note: Wind speed in miles per hour.

**Figure 3-79: Maximum 10-minute Winds near WSES for Storm 302**



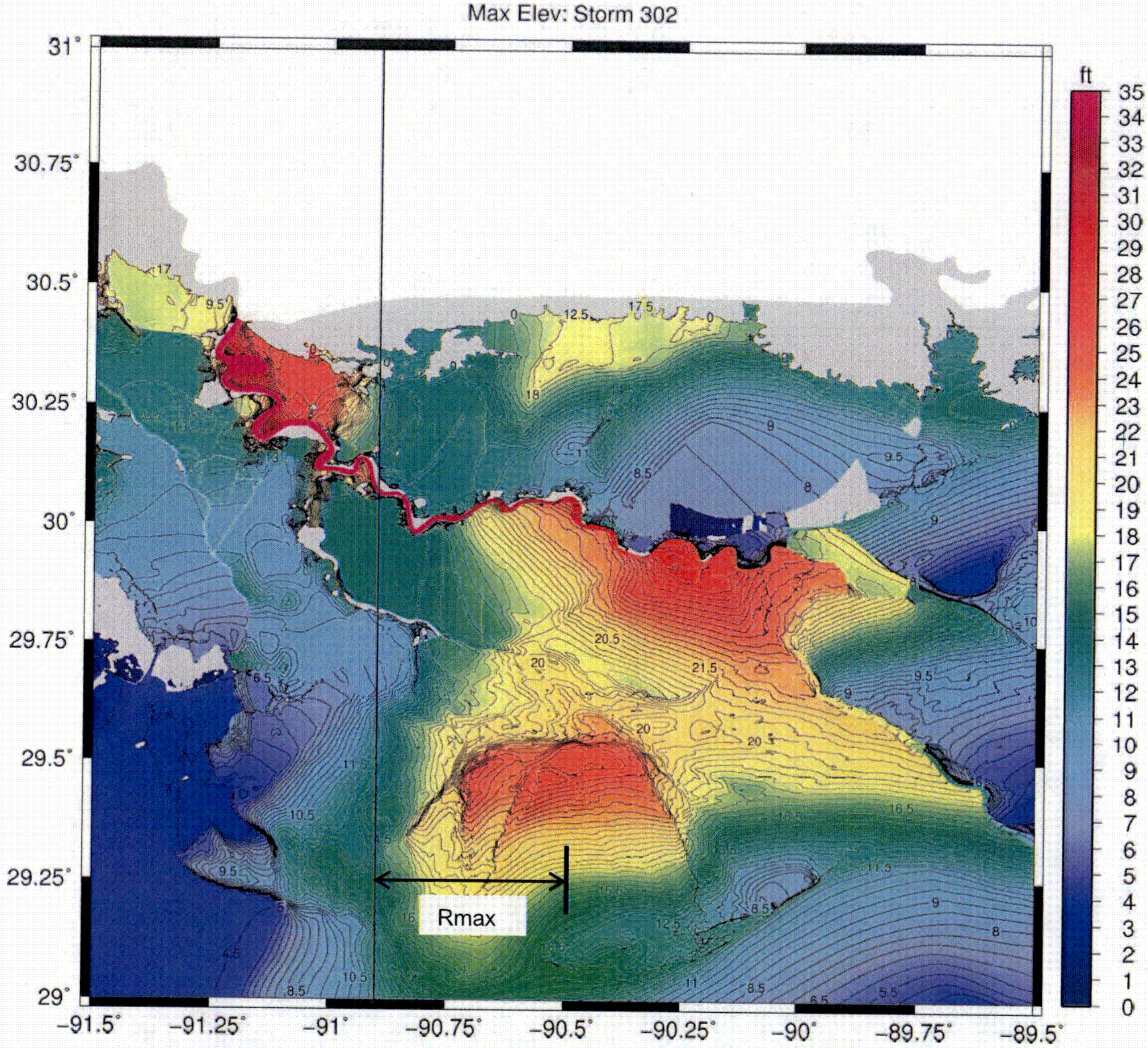

Note: Wind speed in miles per hour.

**Figure 3-80: Maximum 10-minute Winds near WSES for Storm 402c**




Note: Wind speed in miles per hour.

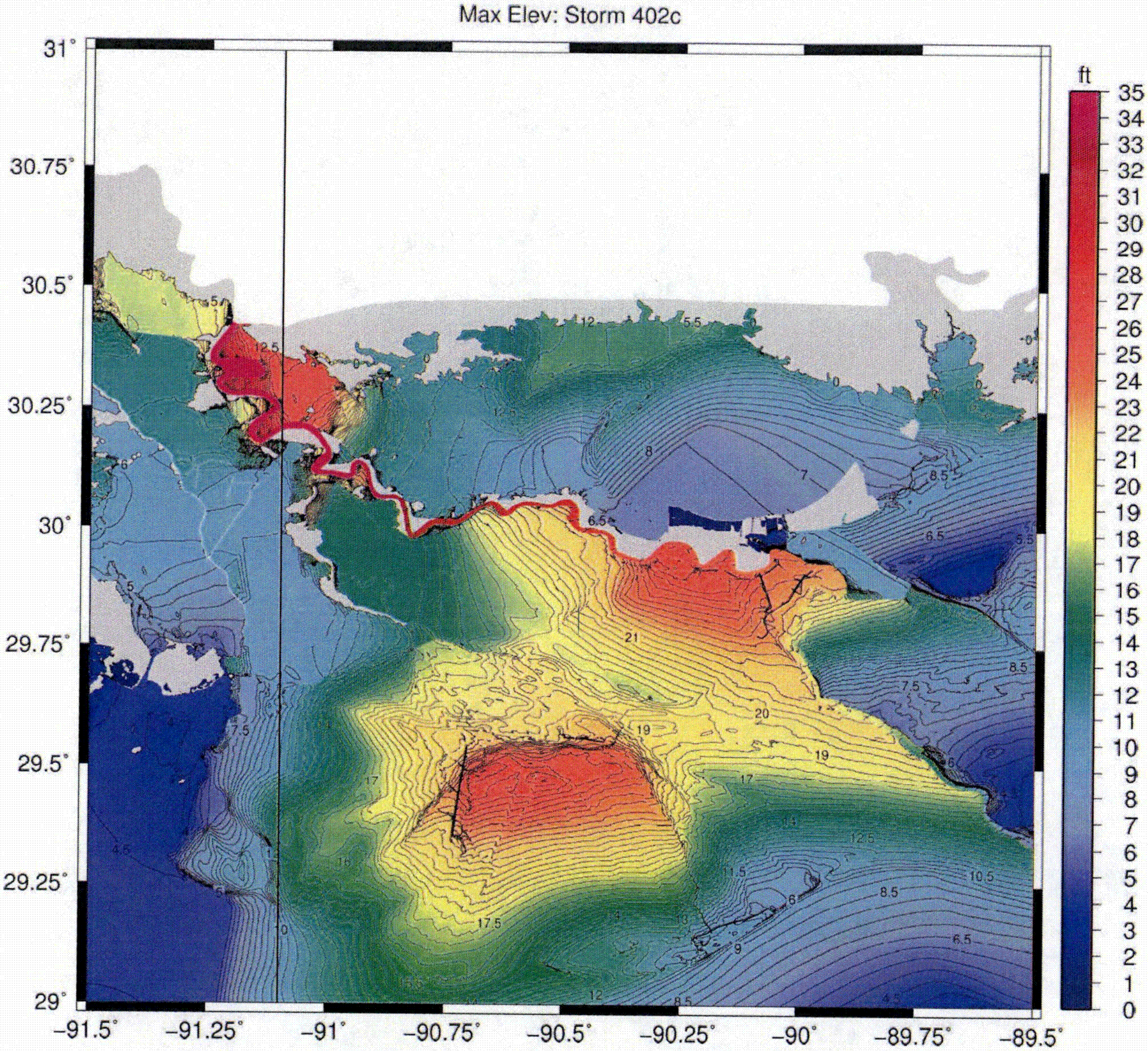
**Figure 3-81: Maximum Unadjusted Water Surface Elevations for Storm 302**




Any illegible text or features in this figure are not pertinent to the technical purposes of this document.

Notes: Elevations are in ft, NAVD88 (2004.65)

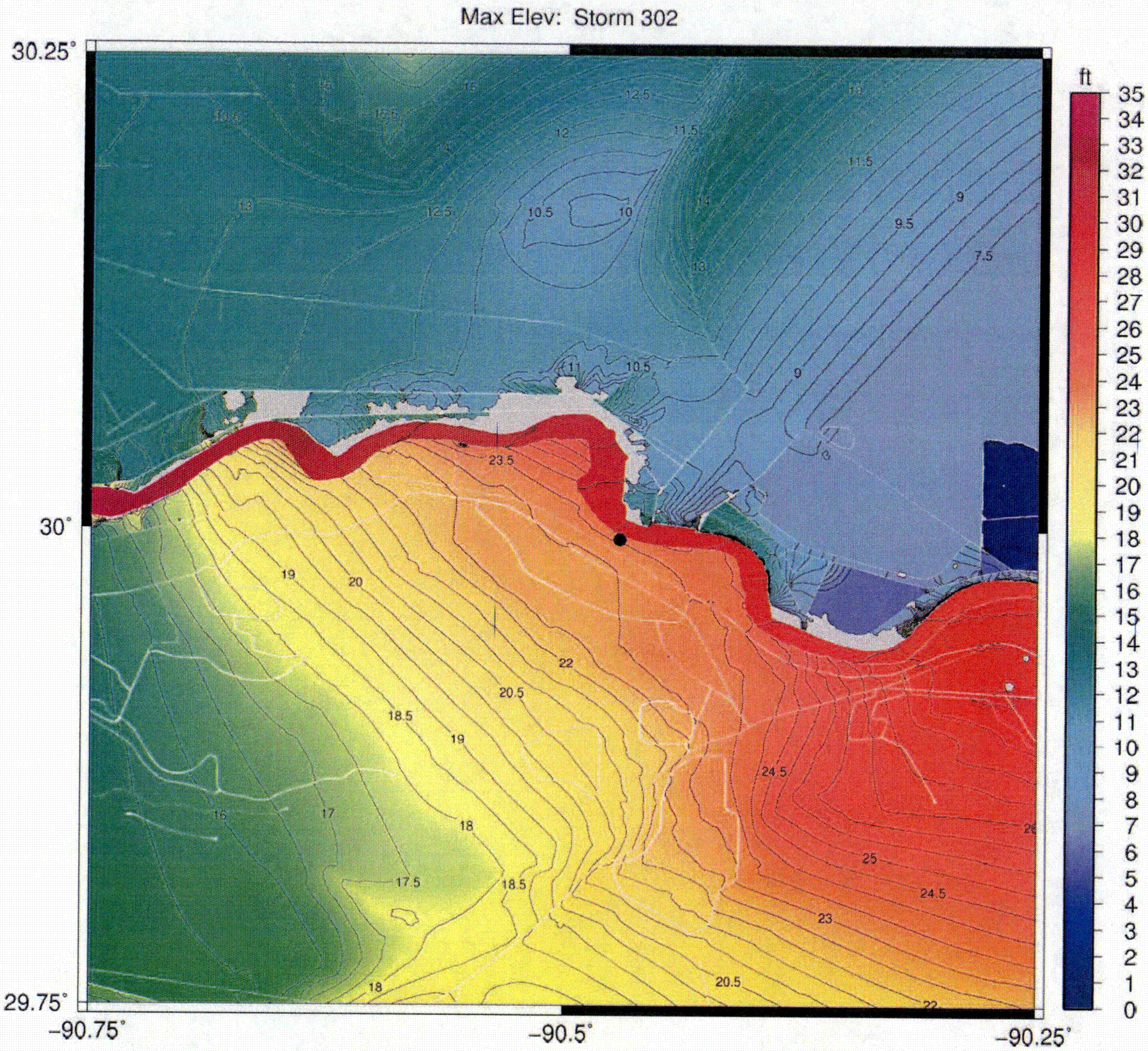
**Figure 3-82: Maximum Unadjusted Water Surface Elevations for Storm 402c**




Any illegible text or features in this figure are not pertinent to the technical purposes of this document.

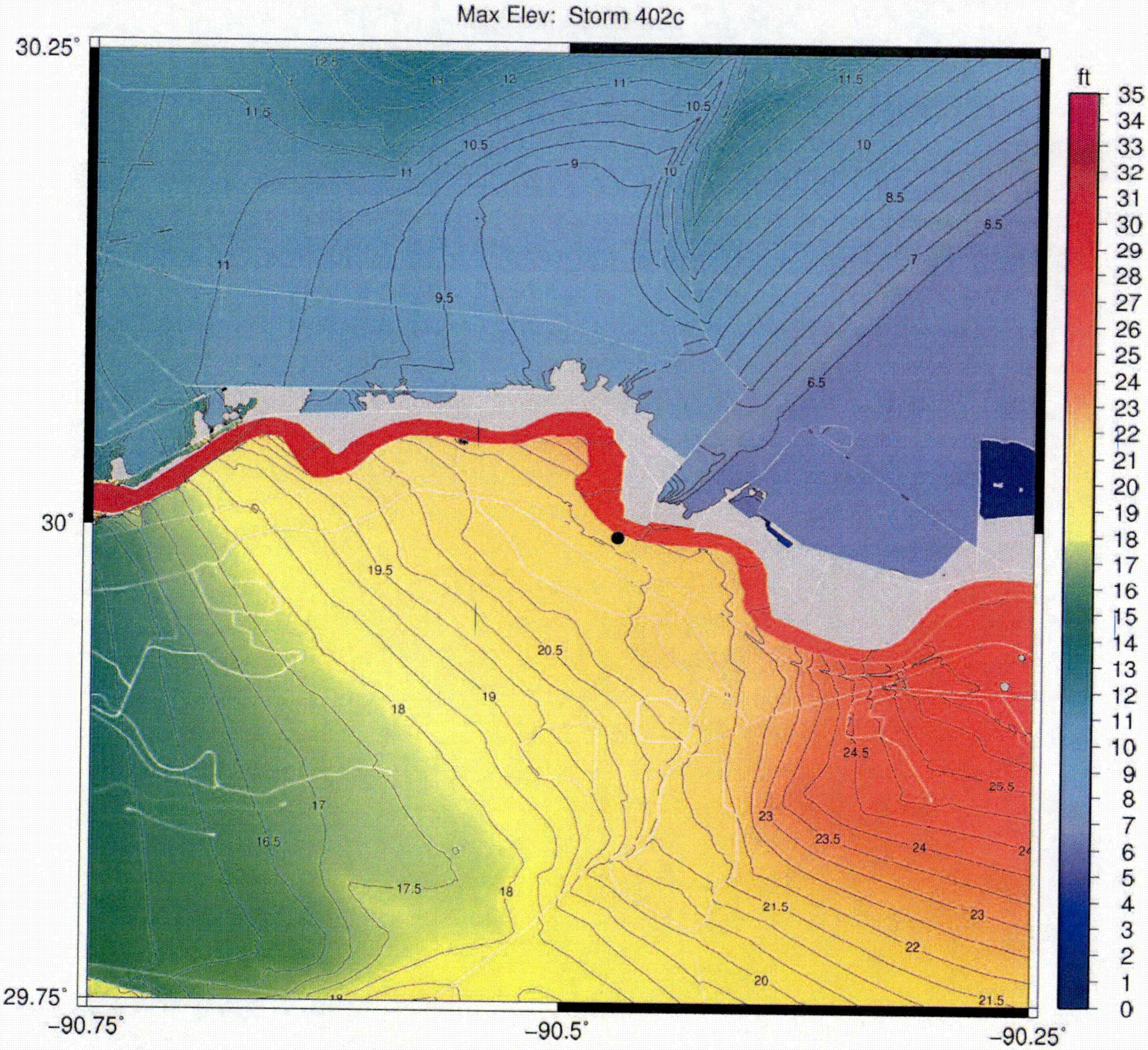
Notes: Elevations are in ft, NAVD88 (2004.65)

**Figure 3-83: Maximum Unadjusted Water Surface Elevations near WSES for Storm 302**



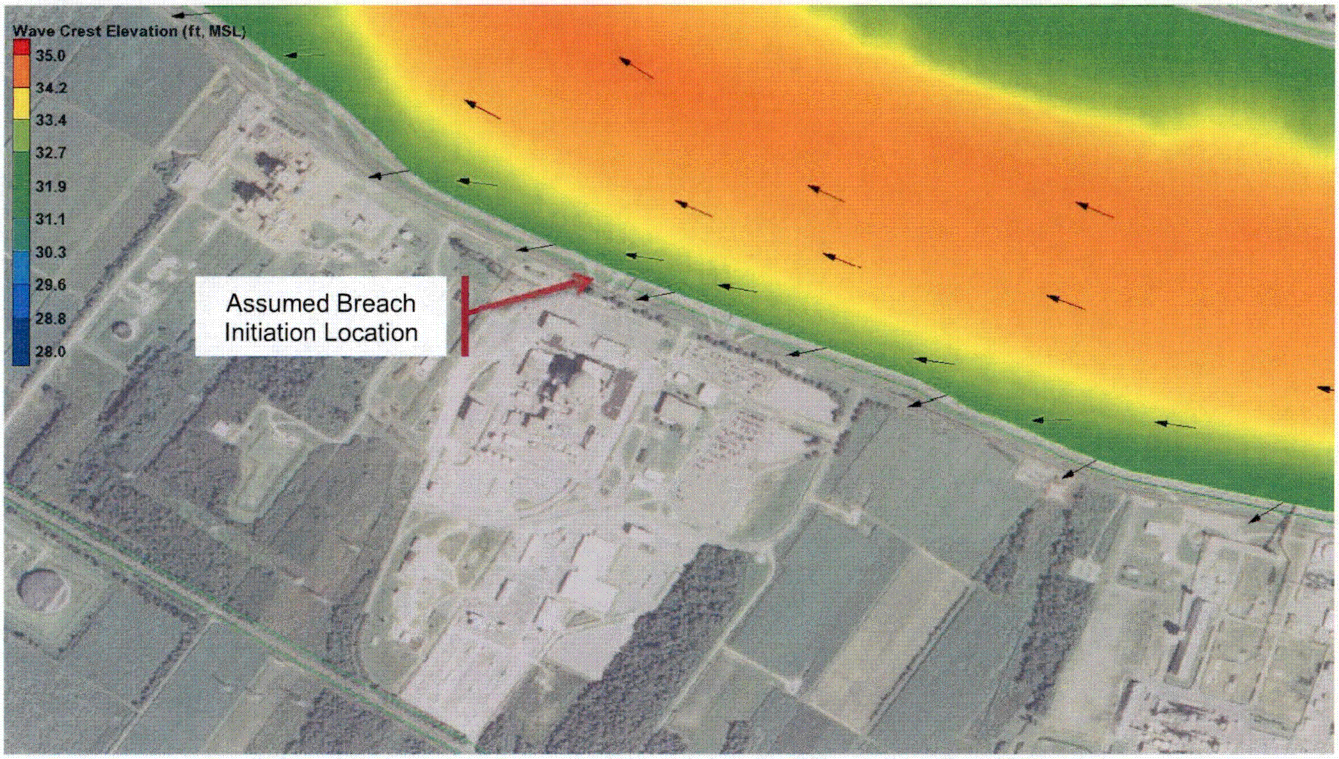

Notes: Elevations are in ft, NAVD88 (2004.65)

**Figure 3-84: Maximum Unadjusted Water Surface Elevations near WSES for Storm 402c**



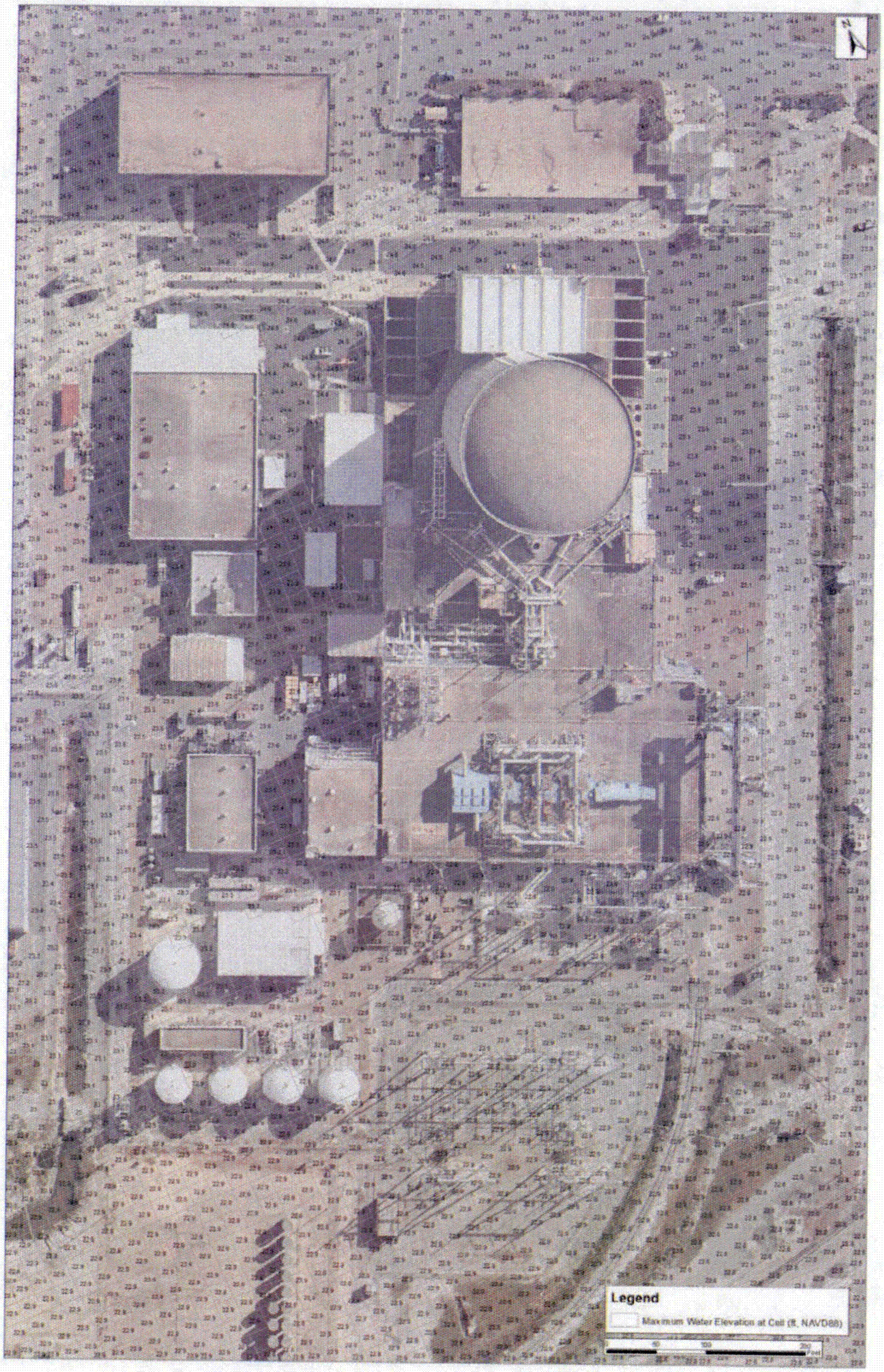

Notes: Elevations are in ft, NAVD88 (2004.65)

**Figure 3-85: Mississippi River Wave Crest Elevation and Wave Direction Map (Storm 402c)**



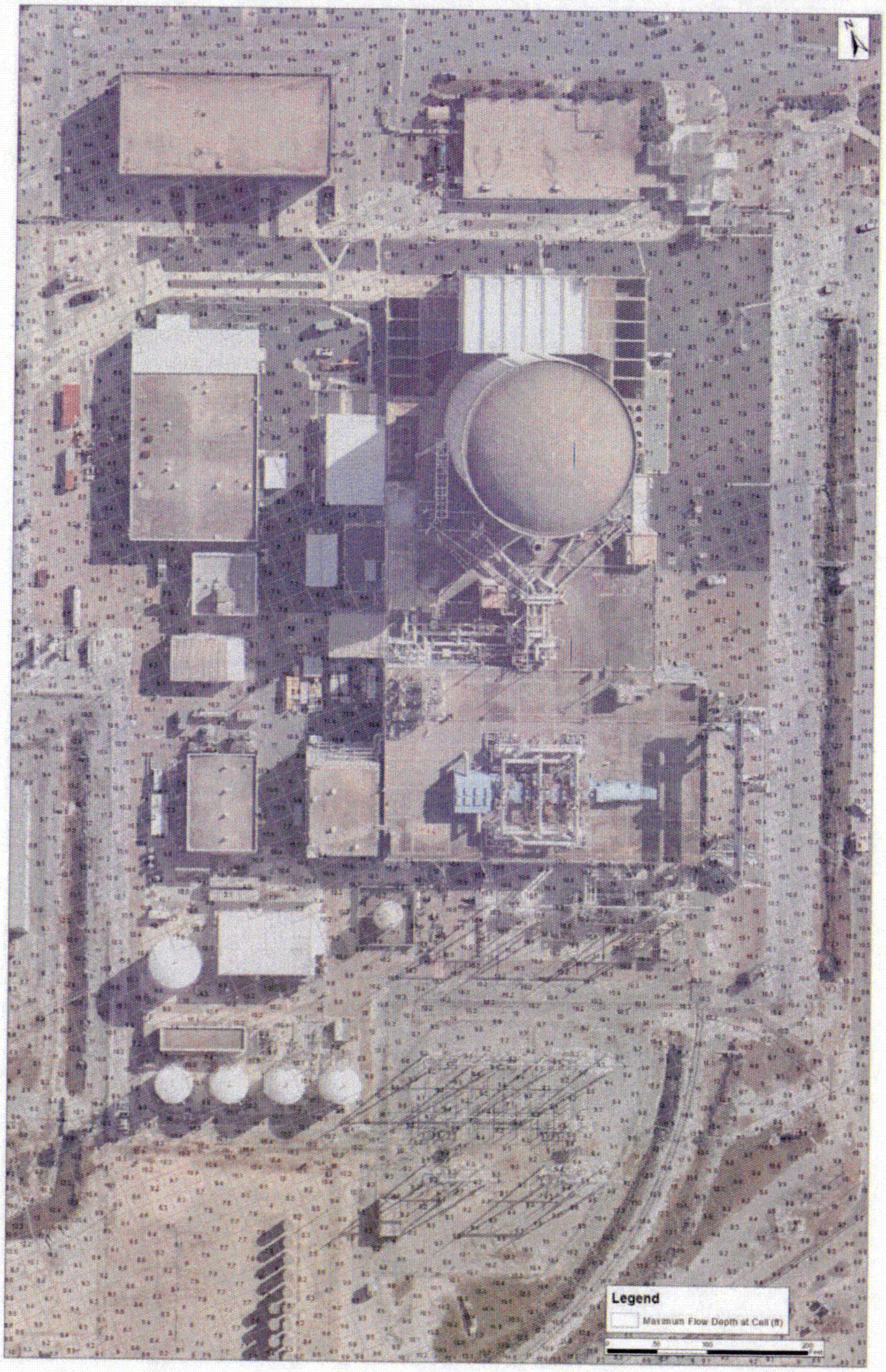

Any illegible text or features in this figure are not pertinent to the technical purposes of this document.

**Figure 3-86: FLO-2D - H.3 (402c) Maximum Water Surface Elevation near NPIS**



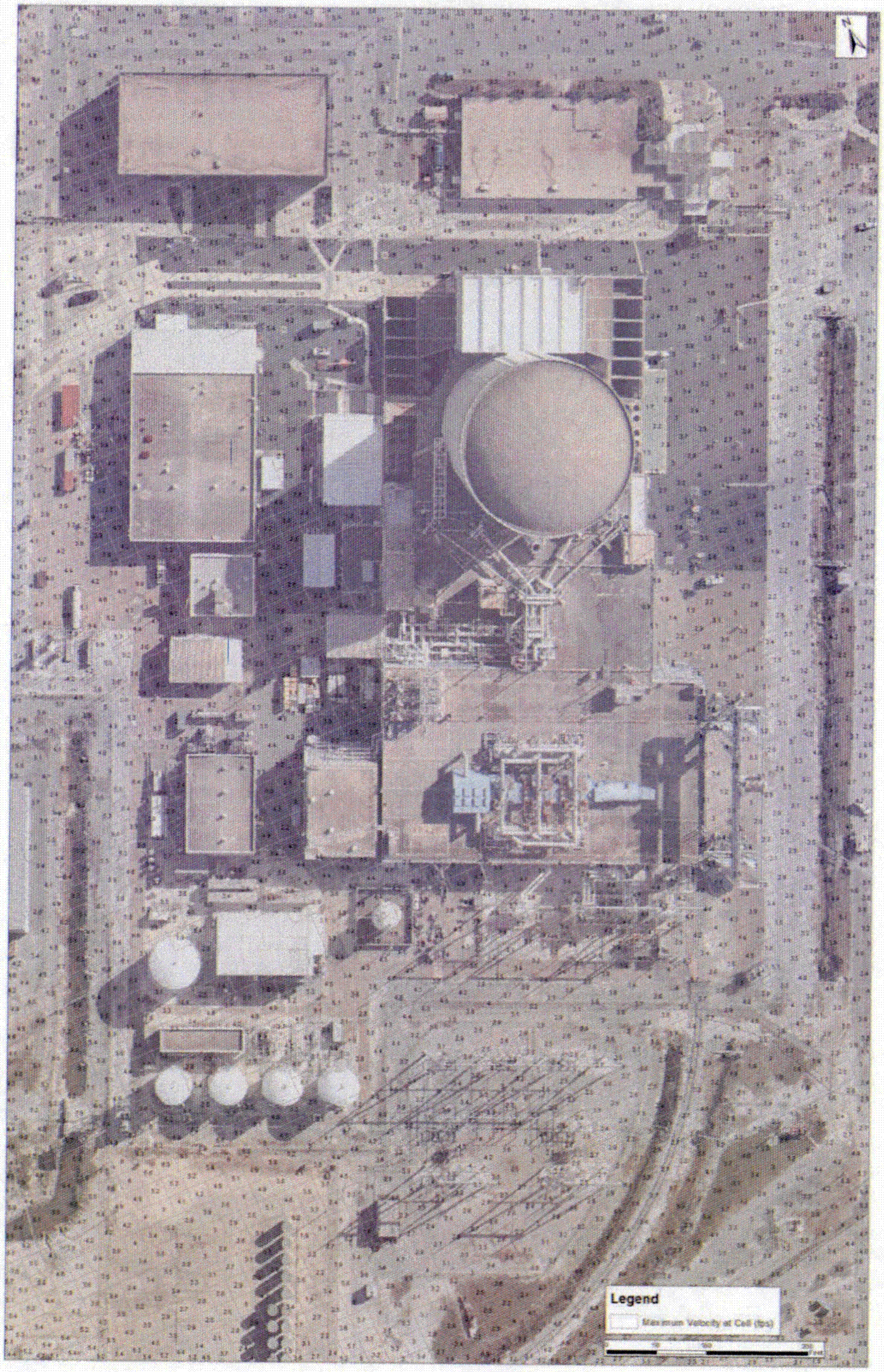

Elevations given in NAVD88-2004.65.
Any illegible text or features in this figure are not pertinent to the technical purposes of this document.

**Figure 3-87: FLO-2D - H.3 (402c) Maximum Flow Depth near NPIS**




Any illegible text or features in this figure are not pertinent to the technical purposes of this document.

**Figure 3-88: FLO-2D - H.3 (402c) Maximum Velocity near NPIS**




Any illegible text or features in this figure are not pertinent to the technical purposes of this document.

**Figure 3-89: FLO-2D - H.3 (402c) Maximum Velocity Vectors near NPIS**


Any illegible text or features in this figure are not pertinent to the technical purposes of this document.

**Figure 3-90: Time Series of Standing Wave Crest Elevations for Storm (302) at WSES**

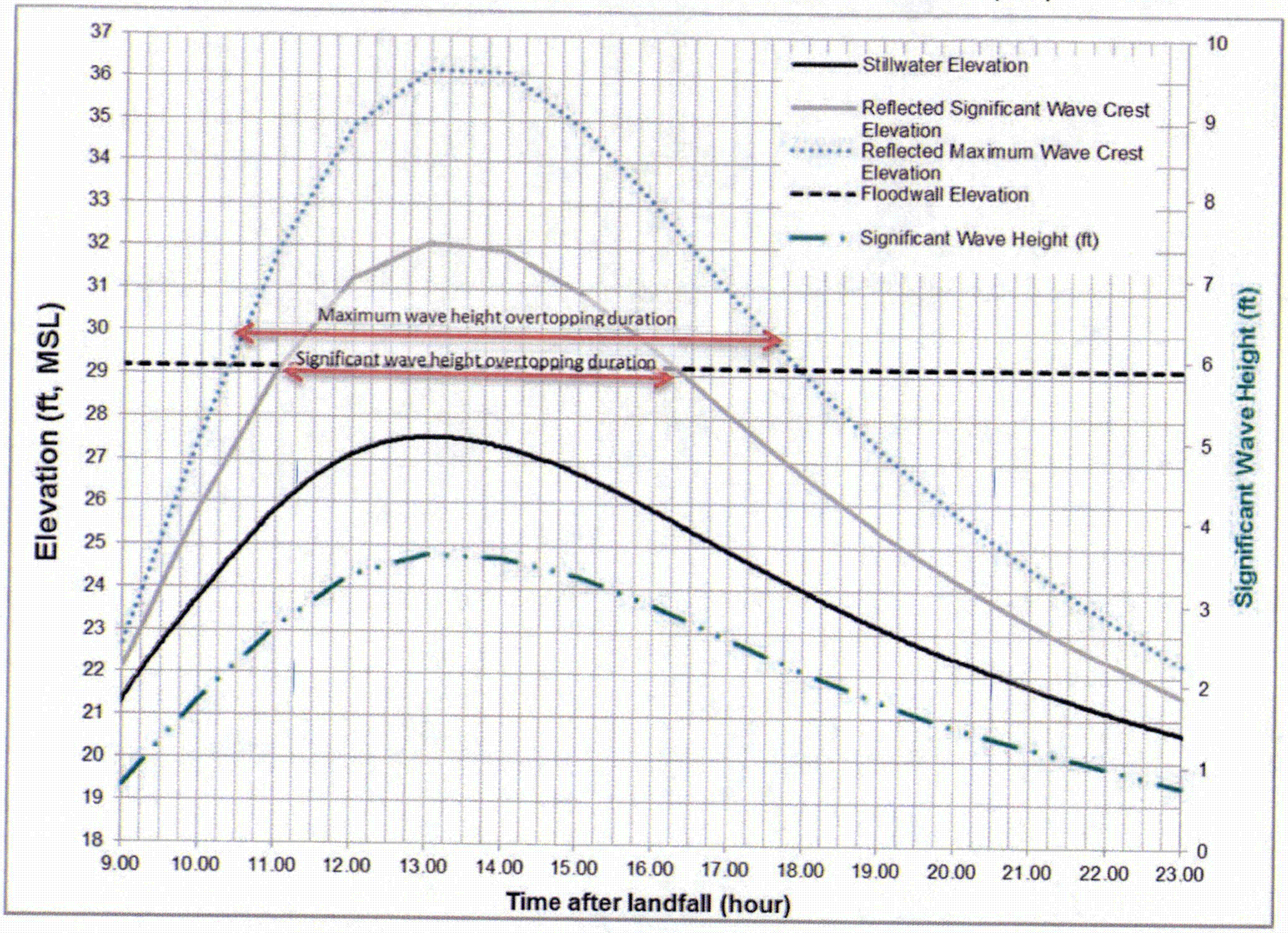
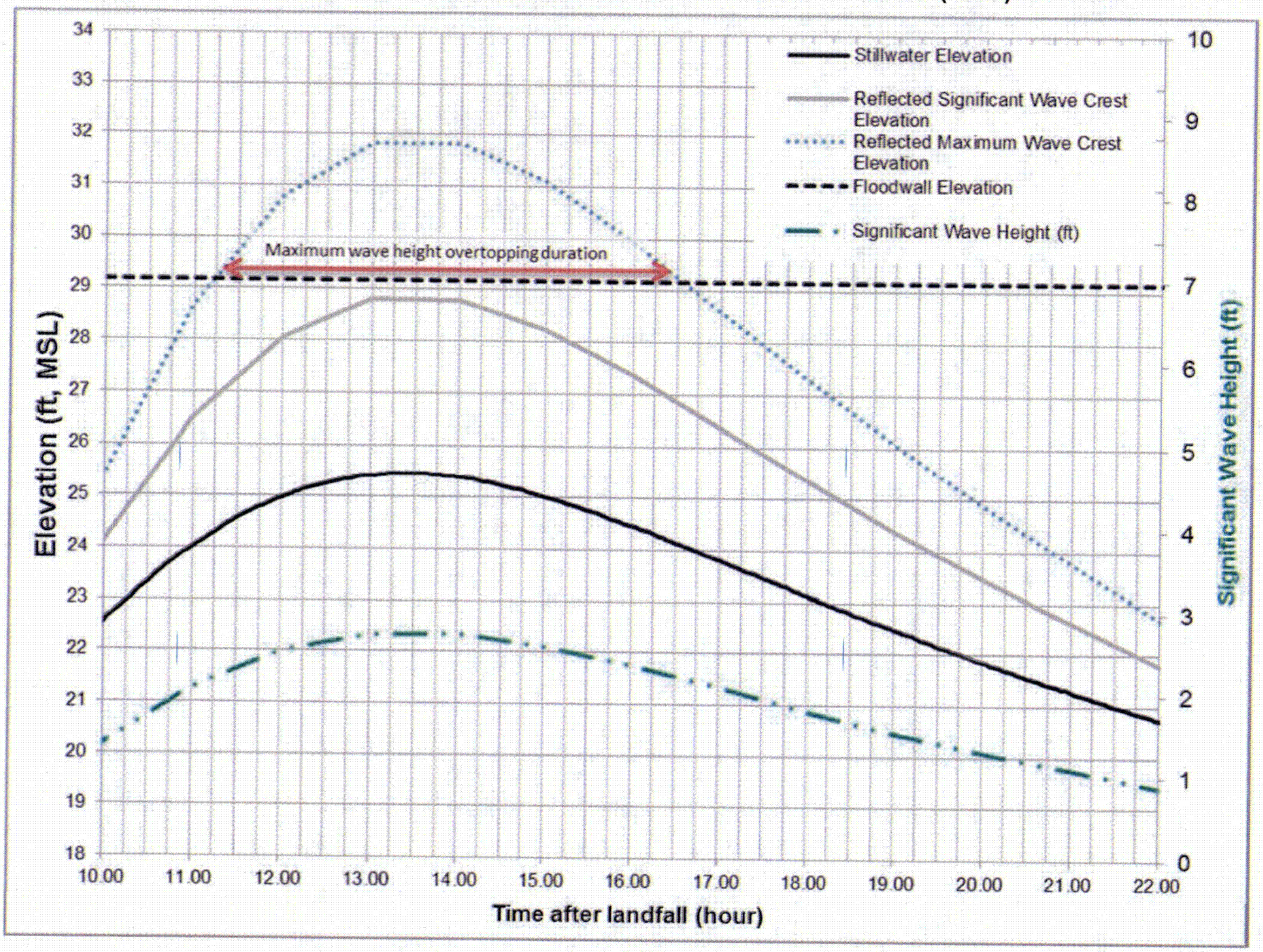


**Figure 3-91: Time Series of Standing Wave Crest Elevations for Storm (402c) at WSES**



## 4.0 FLOOD PARAMETERS AND COMPARISON WITH CURRENT LICENSING BASIS

Per the March 12, 2012, 50.54(f) letter (NRC, 2012a), Enclosure 2, the following flood-causing mechanisms were considered in the flood hazard reevaluation for WSES.

1. Local Intense Precipitation;
2. Flooding in Streams and Rivers;
3. Dam Breaches and Failures;
4. Storm Surge;
5. Seiche;
6. Tsunami;
7. Ice Induced Flooding; and
8. Channel Migration or Diversion.

Some of these individual mechanisms are incorporated into alternative 'Combined Effect Flood' scenarios per Appendix H of NUREG/CR-7046 (NRC, 2011).

Based on the results of the flood hazard re-evaluation, the WSES design basis flood level and associated design basis protections are challenged by two flood mechanisms. The direct precipitation and rooftop drainage during the LIP event results in ponding in the DCT Basins, with potential to impact UHS MCCs. Additionally, the WSES design basis is challenged by flooding due to the combined effects of a PMH-induced PMSS propagating from the GoM towards the site, with coincident 25-year levels in the Mississippi River, 10% exceedance high tide, levee failure at the site, and wind-generated waves (Controlling Combined Effect Flood, CCEF). All other flood mechanisms evaluated for WSES do not threaten the design basis flood level or associated protection.

### 4.1 Summary of Current Licensing Basis and Flood Reevaluation Results

A comparison of the CLB elevations and the reevaluated flood elevations is provided in Table 4-1.

Screened mechanisms have been evaluated at a high level and determined to not be applicable to the flooding hazard for WSES.

Impacts of the flooding due to LIP and the CCEF are addressed in Section 5.0.

### 4.2 References

**NRC, 2011**. "NUREG/CR-7046: Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America", U.S. Nuclear Regulatory Commission, Springfield, VA, National Technical Information Service, 2011.

**NRC, 2012a**. "Request for Information Pursuant to Title 10 of the Code of Federal Regulations 50.54(F) Regarding Recommendations 2.1, 2.3, and 9.3, of the Near-Term Task Force Review of Insights from the Fukushima Dai-Ichi Accident", U.S. Nuclear Regulatory Commission, March 2012.

**NRC, 2012b**. "JLD-ISG-2012-05, Guidance for Performing the Integrated Assessment for External Flooding, Interim Staff Guidance", Revision 0, 2012. (ADAMS Accession No. ML12311A214)

**Table 4-1: Flood Elevation Comparison**

| Mechanism | Design Basis Flood Height | Reevaluated Flood Height | Difference |
|---|---|---|---|
| Local Intense Precipitation | Not evaluated for surface runoff; less than 1.6 ft DCT ponding. | 20.5 ft MSL in WSES yard (1.1 ft depth); 1.53 ft (DCT A) and 1.63 ft (DCT B) ponding[1] | N/A; +0.03 ft DCT ponding |
| PMF on Mississippi River | 27.0 ft MSL in Mississippi River | 29.9 ft MSL in Mississippi River | +2.9 ft |
| Mississippi River PMF + Hypothetical Dam Break | No Hazard | 29.9 ft MSL in Mississippi River Channel, 20.6 ft at WSES. | N/A |
| PMSS from Gulf of Mexico | 18.1 ft MSL | 21.6 ft MSL | +3.5 ft |
| Seiche | No Hazard | No Hazard | N/A |
| Tsunami | No Hazard | No Hazard | N/A |
| Ice-Induced Flooding | No Hazard | No Hazard | N/A |
| Channel Migration or Diversion | No Hazard | No Hazard | N/A |
| Controlling Combined Effect Flood Scenario | 27.6 ft MSL Stillwater at WSES site **[Design Basis Flood Level]** | 26.0 ft MSL Stillwater at WSES, 26.9 ft MSL significant wave crest at WSES, 31.8 ft MSL maximum wave crest, 0.1 cfs overtopping from maximum waves. | -1.6 ft Stillwater, -0.7 ft significant wave crest, +4.2 ft maximum wave crest, 0.1 cfs overtopping. |

Notes:

[1]Assuming existing pumping criteria (see Section 5)

N/A indicates this specific mechanism or combination of mechanisms was not evaluated.

## 5.0 INTERIM EVALUATION AND ACTIONS TAKEN OR PLANNED

Flooding due to Local Intense Precipitation and the CCEF are the only flood mechanisms which challenge the design basis of the WSES site. In response to the re-evaluated flood elevations resulting from the LIP and the CCEF events, an evaluation was performed to determine the impact of inundation at the affected locations.

## 5.1 LIP Flooding in DCT Basins

### 5.1.1 Potential Impacts of LIP Flooding in DCT Basins

The lowest, vulnerable equipment in the DCT Basins are the sump pump motors, which are located at 1.513 ft. above the floor in the DCT B and 1.417 ft. above the floor in the DCT A. Sump pumps are installed in the DCT Basins to mitigate ponding levels from inundation of water. The current design basis at WSES credits one motor driven sump pump activating 30 minutes after the onset of the PMP event, and a diesel powered sump pump activating 3 hours after the onset of the PMP event. Inside both DCT Basins, there are MCCs for the UHS that are important to safety. Based on the current pumping requirements for the DCT Basins, ponding in both DCT Basins would exceed the level of the Sump Pumps, which would result in potential ponding above the design basis, impacting the MCCs.

### 5.1.2 Actions Taken Due to LIP Flooding in DCT Basins

Based on the results of the LIP evaluation, if two sump pumps are activated 30 minutes after the onset of the LIP event, the ponding levels in both DCT Basins are below the height of the MCCs as well as the sump pump motors. Therefore, a change in pump starting time was determined to be an interim action necessary to mitigate the LIP event. An Operational Decision Making Issue (ODMI) implementation action plan was developed to ensure two sump pumps are activated within 30 minutes after the onset of a LIP event. The ODMI utilizes warning time triggers and weather monitoring guidance provided in the Nuclear Energy Institute (NEI) white paper entitled "Warning Time for Maximum Precipitation Events." Utilizing severe weather triggers and weather monitoring will allow WSES Operations Department to begin activities necessary to ensure two sump pumps for each DCT Basin are operating within 30 minutes after the onset of the LIP event. In addition, the LIP analysis credits drains on the adjacent rooftops. The roof drains were walked down by WSES Engineering Staff to determine the extent of debris accumulation per CR-WF3-2015-1282 CA 12. Since these rooftops are above any tree line or other potential sources of debris, there is reasonable assurance there will not be any significant debris blockage of the drains. Additionally, PMRQ 8620-07 was created to inspect and clean critical roof drains on an annual basis to ensure they are clear of debris.

## 5.2 Controlling Combined Effect Flooding

### 5.2.1 Potential Impacts of Controlling Combined Effect Flooding

Flooding due to the CCEF results in a stillwater level of 26.0 ft MSL on the west NPIS wall, and 25.4 ft MSL on the east NPIS wall. These flood levels are bound by the design basis stillwater level of 27.6 ft MSL, and below the NPIS minimum protection height of 29.18 ft MSL.

Due to the configuration of the site and the CCEF, wave propogation will only crest against the WSES site on the east side of the NPIS. The significant wave crest is 26.9 ft MSL, also below the bounding design basis stillwater level. Maximum wave crest elevation is 31.8 ft MSL, and would result in overtopping of the DCT B area at a rate of 0.1 cfs. This overtopping rate and resulting ponding in the DCT B area is bound by precipitation induced flooding at this location.

Hydrodynamic loading from the CCEF is considered bounded by the design basis flood, which has a higher stillwater level. The low flow velocity of the CCEF event would limit the potential for significant debris load impact force on the NPIS.

There are no anticipated impacts to SSCs important to safety due to the CCEF.

### 5.2.2 Actions Taken Due to Controlling Combined Effect Flooding

There are no interim actions required for mitigation of the Controlling Combined Effect Flood.

## 5.3 References

**WSES, 2015.** WSES Response to FHE RFI 2015-001, 2015, See AREVA Document No. 38-9243507-000.

## 6.0 ADDITIONAL ACTIONS

No additional actions are necessary.

## APPENDIX A: ELEVATION DATUM CONVERSION

Waterford Steam Electric Station External Flooding Hazard Re-evaluation
Summary of Datum Conversions

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Datum conversions between NAVD88(2004.65), NAVD88(2009.55) and Mean Sea Level or MSL (used as and sometimes referred to as "Plant Datum") are required to complete the flood hazard re-evaluation calculations at WSES. NAVD88 refers to the North American Vertical Datum of 1988.

**Background**

Datum conversions to and from MSL and the NAVD88 datums are needed to compare flood re-evaluation results to existing design basis elevations at WSES. Elevations in the WSES FSAR (WSES, 2013) are referenced to MSL, a site specific datum. Elevations in the topographic survey of WSES (AREVA, 2014) are referenced to NAVD88 (2009.55). Elevations in the computer model inputs used for the storm surge analysis and other calculations, as well as publicly available elevation data are typically referenced to NAVD88(2004.65). Due to natural subsidence, benchmarks in portions of the Gulf Coast are unreliable in estimating elevations relative to the NAVD88 vertical datum. To solve this problem, the National Geodetic Survey (NGS) sponsored the installation of a network of Continuously Operating Reference Stations (CORS) to facilitate more accurate and precise differential elevation measurements. NGS periodically re-observes survey control monuments in Southern Louisiana and publishes new NAVD88 elevations (referred to as a Height Modernization Project). The Height Modernization Project results are adjusted using the most recent GEOID and the updated elevations are published to represent the most current and most accurately defined benchmark elevations above or below the NAVD88 datum. Each update is designated with an epoch of NAVD88. References to NAVD88 in the region therefore include the "epoch" (i.e. NAVD88-2004.65, NAVD88-2006.81 and NAVD88-2009.55).

**Datum Conversion**

Conversion between the datums is based on survey readings at benchmark K393 (DJ9360) (Benchmark BM U 300 as shown on Drawing Numbers 1, 2 and 3). The proposed resolution of the datum conversion is detailed as follows:

1. NAVD88 (2009.55) will be assumed to be equal to NAVD88 (2004.65) based on information contained in Attachment 1 and summarized in Table 1. NAVD88 (2009.55) is based on Geoid 12A (Attachment 2) and NAVD88 (2004.65) is based on Geoid 03 (last paragraph of page 1 of Attachment 3). The elevation difference between Geoid 12A and Geoid 03 is insignificant (0.025ft).

2. The conversion from NAVD88 (2004.65) to MSL (Plant Datum) is calculated as +1.43 feet. This is based on the information provided by Landmark Surveying, Inc. regarding the elevation of BM U 300. The elevation of the benchmark is stated as 13.02 ft, MSL (Drawing 3) and 11.59 ft, NAVD88 (2004.65) (Drawing 1). The survey information contained on Drawings 1-3 based on Geoid 03 (Attachment 4). Therefore to convert to

an elevation referenced to MSL (Plant Datum), 1.43 feet must be added to the elevation of the same feature which is referenced to the NAVD88 (2004.65) datum.

The calculated datum conversion rate is consistent with historically recorded regional subsidence.

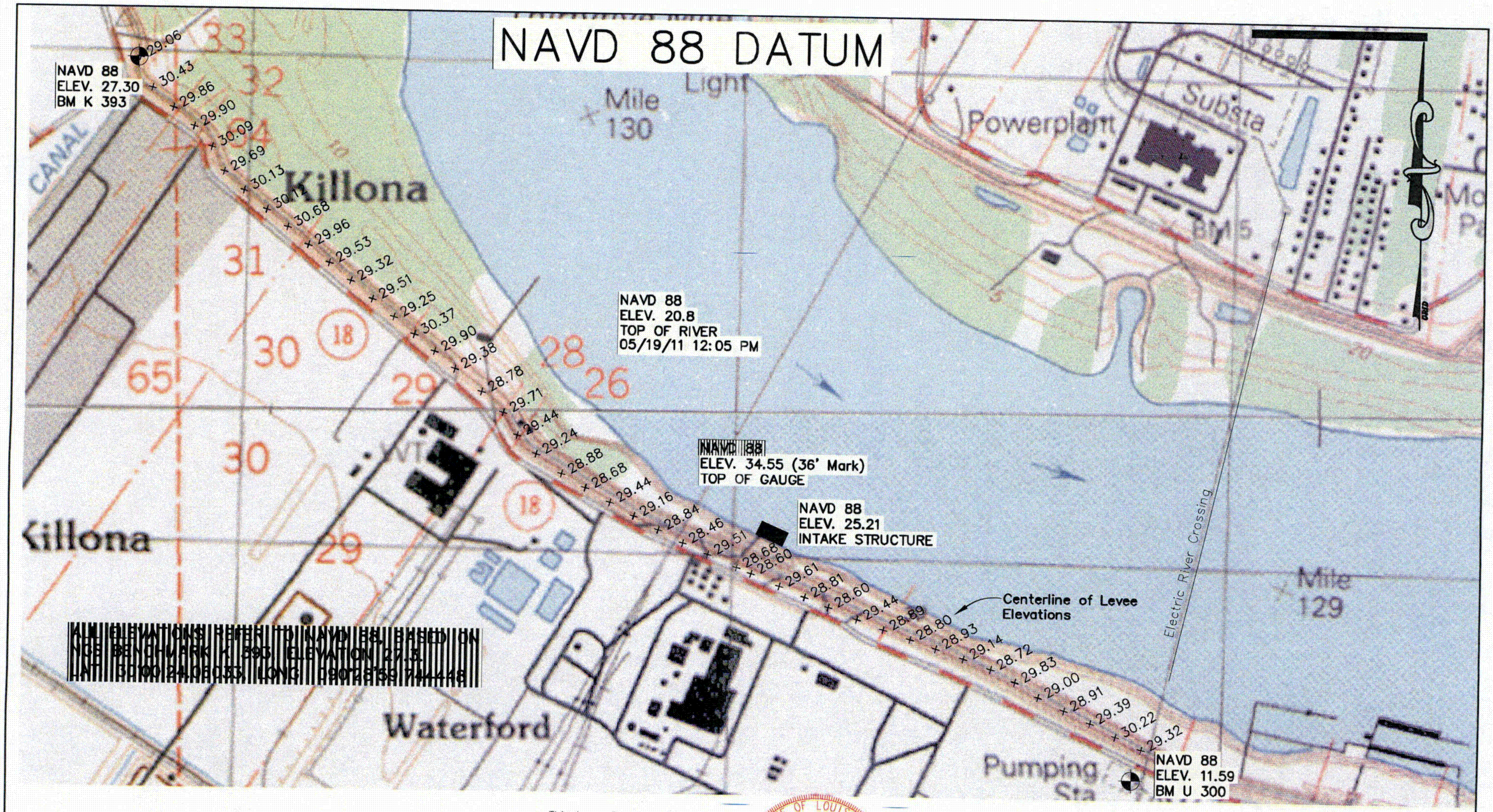
References:

1. **AREVA, 2014**. Waterford Nuclear Generating Station – WSES: Aerial Mapping Validation Report, prepared by McKim & Creed, July 2014. See AREVA Document No. 38-9226991-000.
2. **WSES, 2013**. WSES Updated Final Safety Analysis Report, Revision 307, 2013, See AREVA Document No. 38-9226992-000.
3. **NGS, 2014**. National Geodetic Survey, 2014. http://www.ngs.noaa.gov/

Table 1: Benchmark Elevations in NAVD88 based on different Geoids (NGS, 2014; Attachment 1)

| ID | Elevation | Elevation delta | Elevation delta | Elevation delta |
|---|---|---|---|---|
| | **Geoid12A** | **Geoid12** | **Geoid09** | **Geoid03** |
| DJ9360 (K393) | 8.349 | 8.35 | 8.327 | 8.374 |

# DRAWINGS




| DATE | DESCRIPTION | REVISED |
|---|---|---|
| | REVISIONS | |

GRAPHIC SCALE

600 0 300 600

( IN FEET )
1 inch = 600 ft.

This is an Elevation Survey.
This plat is not a property boundary survey and as such does not comply with the "Louisiana Minimum Standards for Property Boundary Surveys".

STATE OF LOUISIANA
STEVEN M. RUNNEBAUM
REG. NO. 4740
REGISTERED PROFESSIONAL LAND SURVEYOR

PROFESSIONAL LAND SURVEYOR

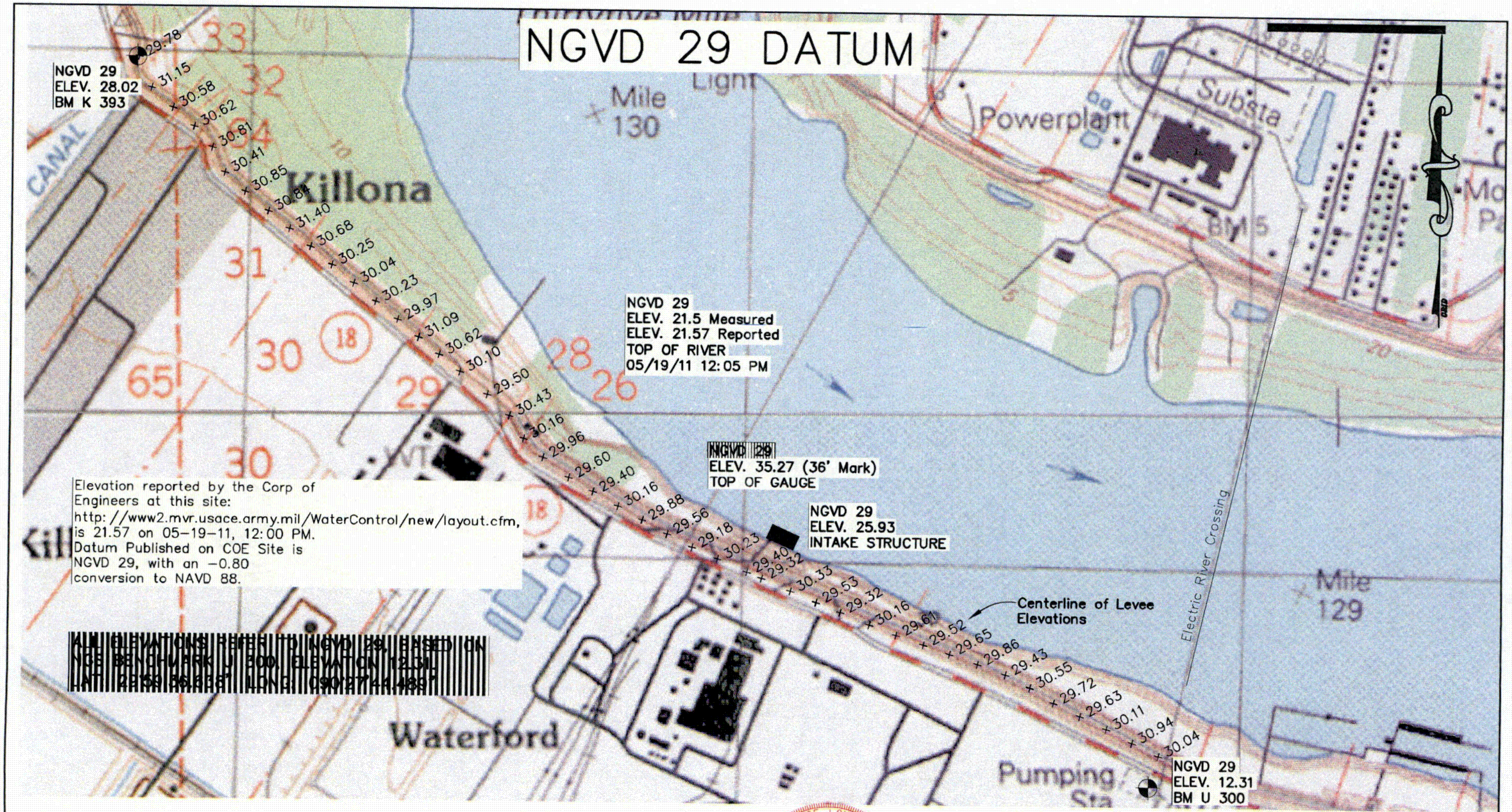
LANDMARK SURVEYING, INC.
1513 Kuebel Street
Harahan, Louisiana 70123
Fax: 504-734-8357
Phone: 504-733-3303

© Copyright

NAVD 88 DATUM
WATERFORD 3 NUCLEAR PLANT
KILLONA
ST. CHARLES PARISH,
LOUISIANA

| DATE: 05/24/11 | DRAWN BY: SMR | JOB NO. | DRAWING NO. |
|---|---|---|---|
| SCALE: 1" = 600' | CHECKED BY: SMR | 1[redacted] | [redacted] of 3 |

PLAT FILE NO: 110155-1




| DATE | DESCRIPTION | REVISED |
|---|---|---|
| | REVISIONS | |

**GRAPHIC SCALE**

600 0 300 600

( IN FEET )
1 inch = 600 ft.

This is an Elevation Survey.
This plat is not a property boundary survey and as such does not comply with the "Louisiana Minimum Standards for Property Boundary Surveys".

STATE OF LOUISIANA
STEVEN M. RUNNEBAUM
REG. NO. 4740
REGISTERED PROFESSIONAL LAND SURVEYOR

PROFESSIONAL LAND SURVEYOR

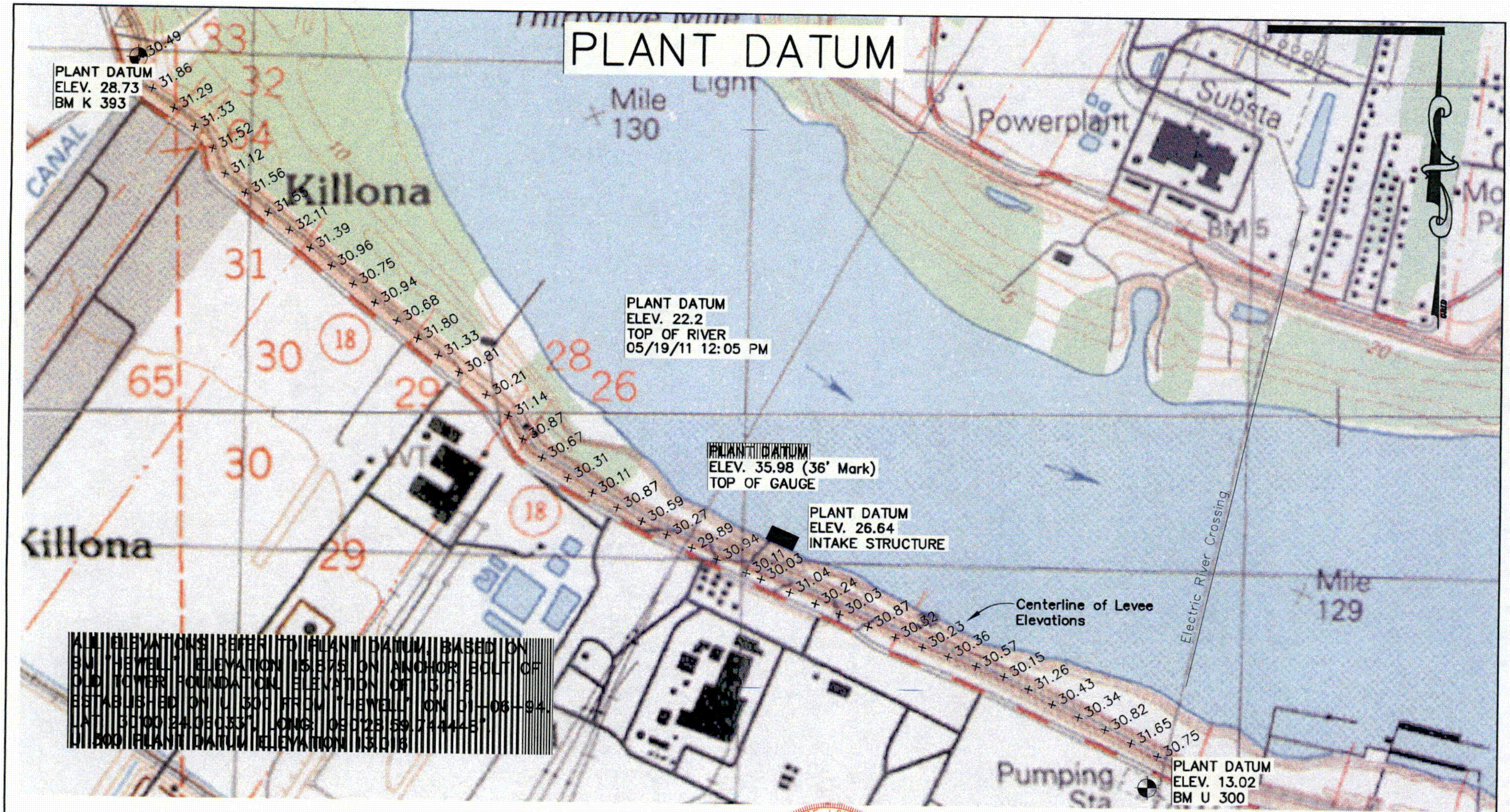
LANDMARK SURVEYING, INC.
1513 Kuebel Street
Harahan, Louisiana 70123
Fax: 504-734-8357
Phone: 504-733-3303



NGVD 29 DATUM
WATERFORD 3 NUCLEAR PLANT
KILLONA
ST. CHARLES PARISH,
LOUISIANA

| DATE: 05/24/11 | DRAWN BY: SMR | JOB NO. | DRAWING NO. |
|---|---|---|---|
| SCALE: 1" = 600' | CHECKED BY: SMR | 1[redacted]2 of 3 | |

PLAT FILE NO: 110155-2




| DATE | DESCRIPTION | REVISED |
|---|---|---|
| | REVISIONS | |

GRAPHIC SCALE
600 0 300 600
( IN FEET )
1 inch = 600 ft.

This is an Elevation Survey.
This plat is not a property boundary survey and as such does not comply with the "Louisiana Minimum Standards for Property Boundary Surveys".

STATE OF LOUISIANA
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Harahan, Louisiana 70123
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Phone: 504-733-3303
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PLANT DATUM
WATERFORD 3 NUCLEAR PLANT
KILLONA
ST. CHARLES PARISH,
LOUISIANA

| DATE: 05/24/11 | DRAWN BY: SMR | JOB NO. | DRAWING NO. |
|---|---|---|---|
| SCALE: 1" = 600' | CHECKED BY: SMR | 1[redacted] | [redacted]3 of 3 |

PLAT FILE NO: 110155-3

# ATTACHMENT 1 – GEOID CONVERSION SHEETS

DJ9360_Geographic_NAD83_2011_Meters_Geoid_12A.txt

```
Input:
        Type:     Geographic
        HzUnits:  Degrees Minutes Seconds
        VtUnits:  Metres
        Height:   ELLIPSOIDAL
        Datum:    NAD83(2011)

Output:
        Type:     Geographic
        HzUnits:  Degrees Minutes Seconds
        VtUnits:  Metres
        Height:   ORTHOMETRIC
        Datum:    NAD83(2011)

Geoid File:       'C:\Installs\GEOID12A(CONUS)\GEOID12A(CONUS).wpg'

ID                  LATITUDE         LONGITUDE      HEIGHT
DJ9360       30 00 24.06028   -90 28 59.74392      8.349
```

Input:

| | |
|---|---|
| Type: | Geographic |
| HzUnits: | Degrees Minutes Seconds |
| VtUnits: | Metres |
| Height: | ELLIPSOIDAL |
| Datum: | NAD83(2011) |

Output:

| | |
|---|---|
| Type: | Geographic |
| HzUnits: | Degrees Minutes Seconds |
| VtUnits: | Metres |
| Height: | ORTHOMETRIC |
| Datum: | NAD83(2011) |

Geoid File: 'C:\Installs\Geoid03-CONUS\Geoid03(CONTUS)CURRENT\GEOID03(CONTUS)CURRENT.wpg'

| ID | LATITUDE | LONGITUDE | HEIGHT |
|---|---|---|---|
| DJ9360 | 30 00 24.06028 | -90 28 59.74392 | 8.374 |

DJ9360_Geographic_NAD83_2011_Meters_Geoid_09.txt

```
Input:
        Type:     Geographic
        HzUnits: Degrees Minutes Seconds
        VtUnits: Metres
        Height:   ELLIPSOIDAL
        Datum:    NAD83(2011)

Output:
        Type:     Geographic
        HzUnits: Degrees Minutes Seconds
        VtUnits: Metres
        Height:   ORTHOMETRIC
        Datum:    NAD83(2011)

Geoid File:       'C:\Installs\Geoid09-CONUS\Geoid09-CONUS.wpg'

ID                LATITUDE         LONGITUDE      HEIGHT
DJ9360      30 00 24.06028  -90 28 59.74392      8.327
```

DJ9360_Geographic_NAD83_2011_Meters_Geoid_12.txt

Input:

```
        Type:     Geographic
        HzUnits:  Degrees Minutes Seconds
        VtUnits:  Metres
        Height:   ELLIPSOIDAL
        Datum:    NAD83(2011)
```

Output:

```
        Type:     Geographic
        HzUnits:  Degrees Minutes Seconds
        VtUnits:  Metres
        Height:   ORTHOMETRIC
        Datum:    NAD83(2011)
```

Geoid File: 'C:\Installs\GEOID12(CONUS)\GEOID12(CONUS).wpg'

| ID | LATITUDE | LONGITUDE | HEIGHT |
|---|---|---|---|
| DJ9360 | 30 00 24.06028 | -90 28 59.74392 | 8.350 |

# ATTACHMENT 2 – BENCHMARK K393 CURRENT SURVEY CONTROL

# The NGS Data Sheet

See file dsdata.txt for more information about the datasheet.

```
PROGRAM = datasheet95, VERSION = 8.5
1        National Geodetic Survey,   Retrieval Date = OCTOBER 30, 2014
 DJ9360 ***********************************************************************
 DJ9360  HT_MOD       -  This is a Height Modernization Survey Station.
 DJ9360  DESIGNATION -  K 393
 DJ9360  PID          -  DJ9360
 DJ9360  STATE/COUNTY-  LA/ST CHARLES
 DJ9360  COUNTRY      -  US
 DJ9360  USGS QUAD    -  LAPLACE (1992)
 DJ9360
 DJ9360                         *CURRENT SURVEY CONTROL
 DJ9360  ___________________________________________________________________
 DJ9360* NAD 83(2011) POSITION- 30 00 24.06028(N) 090 28 59.74392(W)   ADJUSTED
 DJ9360* NAD 83(2011) ELLIP HT-   -17.856 (meters)        (06/27/12)   ADJUSTED
 DJ9360* NAD 83(2011) EPOCH   -  2010.00
 DJ9360* NAVD 88 ORTHO HEIGHT -      8.35  (meters)       27.4   (feet) GPS OBS
 DJ9360* NAVD 88 EPOCH        -  2009.55
 DJ9360  **This station is located in a suspected subsidence area (see below).
 DJ9360  ___________________________________________________________________
 DJ9360  GEOID HEIGHT    -         -26.21  (meters)                     GEOID12A
 DJ9360  NAD 83(2011) X  -    -46,624.364 (meters)                      COMP
 DJ9360  NAD 83(2011) Y  - -5,527,674.075 (meters)                      COMP
 DJ9360  NAD 83(2011) Z  -  3,171,006.399 (meters)                      COMP
 DJ9360  LAPLACE CORR    -           0.37  (seconds)                    DEFLEC12A
 DJ9360
 DJ9360  FGDC Geospatial Positioning Accuracy Standards (95% confidence, cm)
 DJ9360  Type                                       Horiz  Ellip  Dist(km)
 DJ9360  -------------------------------------------------------------------
 DJ9360  NETWORK                                     0.34   1.55
 DJ9360  -------------------------------------------------------------------
 DJ9360  MEDIAN LOCAL ACCURACY AND DIST (117 points)  0.53   2.27     50.48
 DJ9360  -------------------------------------------------------------------
 DJ9360  NOTE: Click here for information on individual local accuracy
 DJ9360  values and other accuracy information.
 DJ9360
 DJ9360
 DJ9360.The horizontal coordinates were established by GPS observations
 DJ9360.and adjusted by the National Geodetic Survey in June 2012.
 DJ9360
 DJ9360.NAD 83(2011) refers to NAD 83 coordinates where the reference
 DJ9360.frame has been affixed to the stable North American tectonic plate. See
 DJ9360.NA2011 for more information.
 DJ9360
 DJ9360.The horizontal coordinates are valid at the epoch date displayed above
 DJ9360.which is a decimal equivalence of Year/Month/Day.
 DJ9360
 DJ9360 ** This station is in an area of known vertical motion.  Due to the
 DJ9360 ** variability of land subsidence, uplift, and crustal motion, NGS has,
 DJ9360 ** determined the orthometric heights for marks in these suspect
 DJ9360 ** subsidence areas should be considered valid only at the epoch date
 DJ9360 ** associated with the orthometric height.  These heights must always
 DJ9360 ** be validated when used as control.  All previously superseded
```

```
DJ9360 ** orthometric heights are now considered suspect and are available
DJ9360 ** in the superseded section.  NGS does not recommend using suspect
DJ9360 ** or superseded heights as control.
DJ9360
DJ9360.The orthometric height was determined by GPS observations and a
DJ9360.high-resolution geoid model using precise GPS observation and
DJ9360.processing techniques.
DJ9360
DJ9360.Photographs are available for this station.
DJ9360
DJ9360.The X, Y, and Z were computed from the position and the ellipsoidal ht.
DJ9360
DJ9360.The Laplace correction was computed from DEFLEC12A derived deflections.
DJ9360
DJ9360.The ellipsoidal height was determined by GPS observations
DJ9360.and is referenced to NAD 83.
DJ9360
DJ9360. The following values were computed from the NAD 83(2011) position.
DJ9360
DJ9360;                     North         East     Units Scale Factor Converg.
DJ9360;SPC LA S     -   167,311.418 1,082,007.863   MT  0.99992575   +0 25 30.2
DJ9360;SPC LA S     -   548,920.88  3,549,887.46   sFT  0.99992575   +0 25 30.2
DJ9360;UTM  15      - 3,322,193.344   742,756.572   MT  1.00032713   +1 15 33.3
DJ9360
DJ9360!             -  Elev Factor  x  Scale Factor =   Combined Factor
DJ9360!SPC LA S     -   1.00000280  x   0.99992575  =   0.99992855
DJ9360!UTM  15      -   1.00000280  x   1.00032713  =   1.00032994
DJ9360
DJ9360                         SUPERSEDED SURVEY CONTROL
DJ9360
DJ9360  ELLIP H (10/11/11)  -17.863  (m)                        GP(        ) 4 1
DJ9360  NAD 83(2007)-  30 00 24.06033(N)    090 28 59.74448(W) AD(2002.00) A
DJ9360  ELLIP H (03/12/08)  -17.854  (m)                        GP(2006.81) 3 1
DJ9360  NAVD 88 (03/12/08)    8.33   (m)  GEOID03 model used    GP(2006.81)
DJ9360
DJ9360.Superseded values are not recommended for survey control.
DJ9360
DJ9360.NGS no longer adjusts projects to the NAD 27 or NGVD 29 datums.
DJ9360.See file dsdata.txt to determine how the superseded data were derived.
DJ9360
DJ9360_U.S. NATIONAL GRID SPATIAL ADDRESS: 15RYP4275622193(NAD 83)
DJ9360
DJ9360_MARKER: F = FLANGE-ENCASED ROD
DJ9360_SETTING: 59 = STAINLESS STEEL ROD IN SLEEVE (10 FT.+)
DJ9360_STAMPING: K 393 2006
DJ9360_MARK LOGO: NGS
DJ9360_PROJECTION: RECESSED 3 CENTIMETERS
DJ9360_MAGNETIC: I = MARKER IS A STEEL ROD
DJ9360_STABILITY: B = PROBABLY HOLD POSITION/ELEVATION WELL
DJ9360_SATELLITE: THE SITE LOCATION WAS REPORTED AS SUITABLE FOR
DJ9360+SATELLITE: SATELLITE OBSERVATIONS - July 26, 2010
DJ9360_ROD/PIPE-DEPTH: 25.3 meters
DJ9360_SLEEVE-DEPTH   :  0.9 meters
DJ9360
DJ9360  HISTORY     - Date     Condition        Report By
DJ9360  HISTORY     - 20060708 MONUMENTED       NGS
DJ9360  HISTORY     - 20090120 GOOD             AERODA
DJ9360  HISTORY     - 20090410 GOOD             WOOLPT
DJ9360  HISTORY     - 20100321 GOOD             JCLS
DJ9360  HISTORY     - 20100726 GOOD             NGS
DJ9360
```

```
DJ9360                          STATION DESCRIPTION
DJ9360
DJ9360'DESCRIBED BY NATIONAL GEODETIC SURVEY 2006
DJ9360'TO REACH FROM I-10 EXIT 220 WEST OF METAIRIE, JUNCTION OF I-310, GO
DJ9360'9.40 MI (15.1 KM) SOUTHWEST ALONG I-310 TO EXIT 10 FOR STATE HIGHWAY
DJ9360'3127, THENCE 0.90 MILE (1.4 KM) SOUTHWEST ALONG EXIT RAMP TO MERGE
DJ9360'WITH HIGHWAY 3127. CONTINUE 7.2 MI (11.6 KM) NORTHWEST ALONG HIGHWAY
DJ9360'3127 TO THE JUNCTION OF STATE HIGHWAY 3141, THENCE RIGHT (NORTHEAST)
DJ9360'1.25 MI (2.0 KM) ALONG HIGHWAY 3141 TO STATE HIGHWAY 18 (RIVER RD),
DJ9360'THENCE LEFT (WEST) 300 FT (90 M) ALONG HIGHWAY 18 TO A GRAVEL ROAD
DJ9360'EAST, THENCE RIGHT (EAST) UP GRAVEL ROAD TO TOP OF LEVEE, THENCE RIGHT
DJ9360'(SOUTHEAST) ALONG LEVEE ROAD ABOUT 70.0 FT (20 M) TO MARK ON RIGHT AT
DJ9360'MILE MARKER 2420.
DJ9360'
DJ9360'MARK IS 164.2 FT (50.05 M) SOUTH AND ACROSS LEVEE ROAD FROM THE SOUTH
DJ9360'ONE OF TWO STEEL GATE POSTS, 143.5 FT (43.7 M) EAST AND UP LEVEE SIDE
DJ9360'SLOPE FROM A POWER POLE WITH A STREET LIGHT, 95.6 FT (29.15 M)
DJ9360'SOUTHEAST OF THE CENTERLINE OF GRAVEL ROAD UP LEVEE, 70.3 FT (21.4 M)
DJ9360'SOUTHEAST OF THE SOUTHERN ONE OF TWO STEEL GATE POSTS, 11.0 FT (33.5
DJ9360'M) SOUTHWEST OF THE CENTERLINE OF LEVEE ROAD, 3.0 FT (0.9 M) NORTHWEST
DJ9360'OF MILE MARKER 2420, AND 0.7 FT (0.2 M) SOUTHWEST OF A CARSONITE
DJ9360'WITNESS POST.
DJ9360'
DJ9360'ACCESS TO MARK IS THROUGH A 5 INCH (13 CM) PVC PIPE AND LOGO CAP.
DJ9360'SLEEVE DEPTH DOES NOT MEET SPECIFICATIONS FOR A CLASS A MARK.
DJ9360
DJ9360                          STATION RECOVERY (2009)
DJ9360
DJ9360'RECOVERY NOTE BY AERO DATA CORPORATION 2009 (RJG)
DJ9360'WITNESS POST AND LOGO CAP BROKEN
DJ9360
DJ9360                          STATION RECOVERY (2009)
DJ9360
DJ9360'RECOVERY NOTE BY WOOLPERT CONSULTANTS 2009 (JPD)
DJ9360'RECOVERED AS DESCRIBED
DJ9360
DJ9360                          STATION RECOVERY (2010)
DJ9360
DJ9360'RECOVERY NOTE BY JOHN CHANCE LAND SURVEYS INC 2010
DJ9360'RECOVERED IN GOOD CONDITION.
DJ9360
DJ9360                          STATION RECOVERY (2010)
DJ9360
DJ9360'RECOVERY NOTE BY NATIONAL GEODETIC SURVEY 2010 (RWA)
DJ9360'RECOVERED AS DESCRIBED.  THERE IS A LOCKED GATE TO THE TOP OF THE
DJ9360'LEVEE WHERE THE MARK IS LOCATED.  THE HIKE IS ABOUT 60 FT (18.3 M).

*** retrieval complete.
Elapsed Time = 00:00:08
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# ATTACHMENT 3 – USACE DATUM POLICY MEMO

CEMVN-ED-S

MAY 1 5 2014

MEMORANDUM FOR SEE DISTRIBUTION

SUBJECT: Addendum to Revised Vertical Control Requirements for USACE Projects (Engineering Division Datum Policy Memo #2)

1. This document provides updates to the following:

a. CEMVN-ED-S, memo, dated 1 December 2008, SUBJECT: Revised Vertical Control Requirements for USACE Projects (Engineering Division Datum Policy Memo #2).

b. The purpose of this memo is to provide updates to Engineering Division Datum Policy Memo #2 (Encl) based on recent changes by the National Geodetic Survey (NGS), publishing a new geoid model and a new NAVD88 epoch (2009.55).

2. Background on Geoid and Change in the Required Project Datum for the New Orleans to Venice (NOV) and Non-Federal Levee Incorporation into NOV (NF):

a. NGS is committed to defining and maintaining the National Spatial Reference System, and providing updated elevations on benchmarks for the nation. NGS periodically re-observes survey control monuments in Southern Louisiana and publishes new NAVD88 elevations (referred to as a Height Modernization Project). These updated elevations are published to represent the most current and most accurately defined benchmark elevations above or below the NAVD88 datum. Each update is designated with an epoch (or time stamp) of NAVD88.

b. NGS also periodically publishes new geoid models in order to improve the accuracy of GPS-derived elevations. These new geoid models are incorporated into each Height Modernization Project, and the resulting NAVD88 elevations are determined relative to this most current geoid model.

c. As referenced in the Engineering Division Datum Policy Memo #2, (Encl), Paragraph 1.a; the datum/epoch that was current when initiating design efforts for the "existing hurricane protection projects" was NAVD88 (2004.65). This datum/epoch was published using the geoid model that was current at that time (GEOID03).

CEMVN-ED-S
SUBJECT: Addendum to Revised Vertical Control Requirements for USACE Projects (Engineering Division Datum Policy Memo #2)

d. NGS published a new geoid model in July 2012 (GEOID12A), and the results of a new Height Modernization Project in March 2013. The Height Modernization results were adjusted using GEOID12A, and were published as NAVD88 (2009.55).

e. The published changes in the NOV-NF project area are too significant to continue referencing the previously stated project datum/epoch of NAVD88 (2004.65). The elevations at project control points also changed in different magnitudes along the project area (from +0.16 feet to -1.05 feet), making attempts to adjust newly collected data back to the project datum/epoch impossible. Therefore, the project datum/epoch for NOV-NF is changed to NAVD88 (2009.55).

3. Updates to Engineering Division Datum Policy Memo #2 to incorporate this change:

a. Reference 2.d shall reflect the memo being dated 23 March 2009, which superseded the 1 December 2008 memo which was originally referenced.

b. See enclosure, first sentence shall read "**The region's vertical control network is periodically readjusted to develop current elevations (e.g. NAVD88 (2003), NAVD88 (2004.65), NAVD88 (2006.81), <u>NAVD88 (2009.55)</u>) in response to subsidence, <u>and to incorporate changes in the geoid model</u>.**"

c. See enclosure, **Paragraph 1.a shall no longer reference New Orleans to Venice as one of the projects referencing NAVD88 (2004.65).**

d. See enclosure, Paragraph 1.a, last sentence shall be updated to read "**Elevations shall be periodically collected and assessed to monitor actual subsidence versus estimated subsidence, <u>as well as to document changes in elevations due to other factors (i.e. new geoid model)</u>".**

e. See enclosure, Paragraph 1.g shall be added as follows: "**New Orleans to Venice (NOV) and Non-Federal Levee Incorporation into NOV Hurricane and Storm Damage Risk Reduction System Project shall use NAVD88 (2009.55) as the project datum and epoch. Elevations shall be periodically collected and assessed to monitor actual subsidence versus estimated subsidence, as well as**

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SUBJECT: Addendum to Revised Vertical Control Requirements for USACE Projects (Engineering Division Datum Policy Memo #2)

**subsidence, as well as to document changes in elevations due to other factors (i.e. new geoid model)**".

4. These changes are effective immediately. An updated (Encl) 1 is enclosed with above referenced changed incorporated.

5. Questions regarding the required vertical control requirements may be addressed to the District Datum Coordinator, Design Services Branch, Mr. Joshua Hardy (x 1852).

Encl
as

MARK R. HOAGUE, P.E.
Chief, Engineering Division

DISTRIBUTION:
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Chief, Programs and Project Management Division
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DEC 0 1 2008

CEMVN-ED-S

MEMORANDUM FOR SEE DISTRIBUTION

SUBJECT: Revised Vertical Control Requirements for USACE Projects (Engineering Division Datum Policy Memo #2)

1. This document supersedes the following:

a. CEMVN-ED, memo, dated 11 April 2002, SUBJECT: Vertical Control and Permanent Benchmarks.

b. CEMVN-ED, memo, dated 20 December 2002, SUBJECT: Vertical Datum Policy.

2. References:

a. CECW-CE, memo, dated 04 December 2006, SUBJECT: Implementation of Findings from the Interagency Performance Evaluation Task Force (IPET) for Evaluating Vertical Datums and Subsidence/Sea Level Rise Impacts on Flood Control, Shore Protection, Hurricane Protection, and Navigation Projects.

b. CECW-CE, Engineer Circular, 1110-2-6065, dated 01 July 2007, SUBJECT: Guidance for a Comprehensive Evaluation of Vertical Datums on Flood Control, Shore Protection, Hurricane Protection, and Navigation Projects.

c. CEMVN-ED-S, memo, dated 3 October 2008, SUBJECT: Assignment of District Datum Coordinator Role and Authority (District Datum Policy Memo #1).

d. CEMVN-ED-S, memo, dated 1 December 2008, SUBJECT: Requirement for Use of Benchmarks for USACE Projects (Engineering Division Datum Policy Memo #3).

3. The purpose of this memorandum is to update the vertical control requirements for USACE projects in order to implement the lessons learned from the IPET report and the resulting directive for project evaluations of vertical datums (Reference 2b).

4. The region's vertical control is periodically readjusted to develop current elevations based on updated datums and epochs (e.g. NAVD88 (2003), NAVD88 (2004.65), NAVD88 (2006.81)) in response to subsidence. Enclosure 1 indicates the appropriate project datum / epoch to use based on project type and location.

5. Design and construction documents must document the estimated rate of subsidence, project benchmarks, seasonal variation of local mean sea level, unit of measure, datum, and gaging

stations used for determination of tidal/hydraulic datum for use in future reevaluations of the vertical datum to be conducted at each scheduled periodic inspection.

6. All design and construction documents shall be certified and signed by the District's Datum Coordinator (DDC) for compliance to this policy and References 2a, 2b and 2d.

7. All surveying activities within the District boundaries shall be coordinated through the DDC. All contracted design work utilizing independent survey collection or independent gaging shall provide a Survey Plan, to be approved by the DDC, prior to data collection. All surveying activities within the District boundaries shall be performed in accordance with the published USACE MVN Minimum Surveying Guidelines which can be found at *http://www.mvn.usace.army.mil/ed/edss/surveyingguidelines.asp.* All collected or obtained survey deliverables shall be routed to Survey Section within 5 working days for quality assurance, database incorporation, and archival in EGIS and the District's archival system (e.g. ProjectWise).

8. Subsequent periodic reevaluations of project reference elevations and related datums shall be included as an integral component of the Periodic Inspection Program, Quality Assurance Program, Dam Safety Program, or Levee Safety Program, as appropriate. The frequency of these reevaluations is a function of estimated magnitude of geophysical changes that could impact flood protection or navigation grades. See enclosure 1 for more details.

9. All spatial data shall be collected relative to the latest datum/epoch with direct geodetic ties to project control such that the elevations can be converted back to the project design datum and epoch. This will provide current elevations to validate the estimated subsidence rate utilized during design. The direct tie to the existing project vertical control will provide the conversion factor for conversion back to the design datum.

10. These policies are effective immediately.

11. Questions regarding the required vertical control requirements may be addressed to the DDC, Design Services Branch, Mr. Josh Hardy (x1852).

Encl

Walter O. Baumy

WALTER O. BAUMY, P.E.
Chief, Engineering Division

DISTRIBUTION:
Chief, CECW-CE
Commander, Mississippi Valley Division
Director, Task Force Hope

(CONT)
CEMVN-ED-S
SUBJECT: Revised Vertical Control Requirements for USACE Projects (Engineering Division Datum Policy Memo #2)

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Chief, Engineering Control Branch

**Enclosure 1: (to CEMVN-ED-S, memo, dated 1 December 2008, SUBJECT: Revised Vertical Control Requirements for USACE Projects (Engineering Division Datum Policy Memo #2))**

**Vertical Control Requirements for USACE Projects (Revised 12 May 2014)**

The region's vertical control network is periodically readjusted to develop current elevations (e.g. NAVD88 (2003), NAVD88 (2004.65), NAVD88 (2006.81), NAVD88 (2009.55)) in response to subsidence, and to incorporate changes in the geoid model. This process affects projects whose construction spans many years and therefore spans several vertical control adjustments. If each phase of construction were to use the latest adjusted elevations, projects would have segments using varying reference surfaces causing an uneven design grades. Regional subsidence and its effect on the design grades of USACE projects necessitates consideration of the vertical control as it relates to project design, construction, inspection, and maintenance.

1. All project design grades shall incorporate the estimated amount of subsidence/sea level rise for the project's lifecycle, 50 years in most cases. With this estimated subsidence and sea level rise factored into design grades, the use of the design's reference elevation throughout project lifecycles will not affect the intended design grade/protection level. Accordingly the following actions are required:

   a. Existing hurricane protection projects (Lake Pontchartrain and Vicinity, Grand Isle and Vicinity, Larose to Golden Meadow, West Bank and Vicinity, Morganza to the Gulf, and Morgan City and Vicinity) shall use the project datum, elevation, and epoch at the time of initiating design efforts (i.e. NAVD88 (2004.65)). Elevations shall be periodically collected and assessed to monitor actual subsidence versus estimated subsidence, as well as to document changes in elevations due to other factors (i.e. new geoid model).

   b. New hurricane protection projects authorized after the date of this policy memo shall use the latest available datum, elevation, and epoch for design and construction (e.g. NAVD88 (2006.81) or newer).

   c. Flood control protection projects shall use the datum, elevation, and epoch consistent with the compatible flow line, where required (e.g. NAVD88 (2004.65) or newer).

   d. Coastal navigation projects design and maintenance shall be referenced to the latest available Mean Lower Low Water (MLLW) elevations published by the National Oceanic and Atmospheric Administration (e.g. MLLW 2002-2006).

e. Mississippi River and Atchafalaya River, LA (excluding Bayous Chene, Boeuf, and Black) navigation projects design and maintenance shall be referenced to the latest approved Low Water Reference Plane (currently Mississippi River LWRP – 2007 and Atchafalaya River LWRP - 2000).

f. All other existing projects not referenced in above paragraphs shall use the project datum, elevation, and epoch at the time of initiating design efforts (i.e. NAVD88 (2004.65)). All other new projects authorized after the date of this policy memo not referenced in above paragraphs shall use the latest available datum, elevation, and epoch for design and construction (e.g. NAVD88 (2006.81) or newer).

g. New Orleans to Venice (NOV) and Non-Federal Levee Incorporation into NOV (NFL) Hurricane and Storm Damage Risk Reduction System Projects shall use NAVD88 (2009.55) as the project datum and epoch. Elevations shall be periodically collected and assessed to monitor actual subsidence versus estimated subsidence, as well as to document changes in elevations due to other factors (i.e. new geoid model).

2. Consideration for subsidence extends to the District's stream gaging program, since gages are also subsiding thereby causing higher water level readings. All automatic gages, including data collection platform (DCP) gages, shall be inspected yearly and gage offsets to the latest available adjustment will be calculated. These offsets will be used to adjust raw gage readings to the desired datum and epoch, as needed, and shall be maintained in a database by Hydraulics and Hydrologic Branch. All staff gages shall document the current elevation upon inspection, and updated gage offset values will be calculated. High water staff gages on the Mississippi River shall be inspected annually. Other high water staff gages shall be inspected every three years. Hydraulics and Hydrologic Branch shall coordinate with USGS, NOAA, the State of Louisiana and other gage operators to facilitate discussions, exchange information, and improve awareness of datums.

3. Subsequent periodic reevaluations of project reference elevations and related datums shall be included as an integral component of the Periodic Inspection Program, Quality Assurance Program, Dam Safety Program, or Levee Safety Program, as appropriate. The frequency of these reevaluations is a function of estimated magnitude of geophysical changes that could impact flood protection or navigation grades. For project design water levels a reevaluation shall be performed by H&H Branch every 10 years or after a major flood or storm event. Any uncertainties in protection levels that are identified during the inspection will also need to be incorporated into any applicable risk/reliability models developed for the project (Ref: EM 1110-2-1619, Risk Based Analysis for Flood Damage Reduction Studies). Details on these periodic reevaluations will be provided in subsequent guidance.

# ATTACHMENT 4 – EMAIL CORRESPONDENCE

## Kenneth Hunu

**From:** BROWN Dan (AREVA) <Daniel.Brown@areva.com>
**Sent:** Wednesday, November 05, 2014 2:14 PM
**To:** David M. Leone; Chad Cox; Kenneth Hunu
**Subject:** FW: Waterford Survey
**Attachments:** Waterfrod 3 Elevation Maps .pdf

FYI.

I have asked McKim & Creed to:

1. Provide the delta between GEOID 03 (see below) and GEOID 12A, which should allow us to convert to "Plant Datum" based on the previous work performed by Landmark Surveying.
2. Provide the delta between NAVD88 2004.65 and 2009.55.

I should hear back from them tomorrow or Friday in terms of item 1 at the very least.

Let me know if you see any pitfalls.

---

**From:** Steve Runnebaum [mailto:srunnebaum@landmarksurveyinginc.com]
**Sent:** Wednesday, November 05, 2014 11:15 AM
**To:** BROWN Dan (EP/PE)
**Cc:** 'PICARD, STEPHEN'
**Subject:** RE: Waterford Survey

Attached are the 3 maps prepared for Bob Bagnetto at Waterford 3 at the time. When the river in 2011 was above the toe of levee at the site and rising, he was getting concerned about the river overtopping the levees. WE ran a profile down the top of levee in NAVD 88 (Geoid 03) based on K393 as shown. The other maps are PURE datum shifts and nothing measured. Bob and I went back in his and out files to the time of construction and found the Plant Datum referenced to something I had and the NGVD 29 was based on a superseded value I believe on the NGS Printout.

We checked into the published elevation of K393 with our GPS and if I remember correctly we hit very close, less than 0.10'. There are only a few published BM's per parish after August 2005 (Katrina) after the NGS did a 7" shift down on ALL SE La BM's.

See how the NAVD 88 Elevations match your LIDAR Survey, should be close or flat

***Steven M Runnebaum, PLS***
***Survey Manager***
***1513 Keubel Street***
***Harahan, La. 70123***
***Phone 504-733-3303 xt 4***
***Fax 504-734-8357***
***Cell 504-416-4435***

LANDMARK SURVEYING, INC.

**From:** BROWN Dan (AREVA) [mailto:Daniel.Brown@areva.com]
**Sent:** Tuesday, November 04, 2014 1:29 PM
**To:** Steve Runnebaum
**Cc:** PICARD, STEPHEN
**Subject:** RE: Waterford Survey

Steve,

I tried calling a couple times but have missed you. I am available until 3 pm (eastern) this afternoon, then locked into meetings for the rest of the day.

To give you a bit of background as to what information I am looking for...

We performed an aerial LIDAR survey of the site to support high resolution flood modeling. The new survey was done in NAVD88, GEOID 2012A. We are noting significant discrepancies (~2.2 ft) between the new surveyed elevations and the site design elevations which are stated in MSL (presumably equivalent to NGVD29?). It appears from the figure attached from your 2011 work that you have done some work normalizing recent survey data to the "Plant Datum" which I was interpreting to represent the design/construction elevations reported in plant licensing documents.

I was hoping that you would be able to provide some insight or documentation about vertical conversion factors you have identified between recent survey data and the "plant datum", so we can incorporate that conversion factor when comparing our flood results based on new survey elevations to plant features listed in their licensing documents. Additionally, if you have any knowledge as to the cause of the difference between the plant datum and the new survey heights (is it purely NGS adjustment, local settlement/subsidence, regional subsidence, etc.) that would also be very helpful.

Thanks,

***Dan Brown***
Scientist IV
Environmental Health & Safety Analysis
AREVA, Inc.
7207 IBM Drive
Charlotte, NC 28262
704 805 2759 office
508 494 6330 cell
daniel.brown@AREVA.com

**From:** PICARD, STEPHEN [mailto:spicard@entergy.com]
**Sent:** Monday, November 03, 2014 1:37 PM
**To:** BROWN Dan (EP/PE)
**Cc:** Steve Runnebaum
**Subject:** RE: Waterford Survey

Certainly. Also, below I included an email he sent me a few months back about these elevations they shot for us. Steve, AREVA may be calling you to inquire about a project you guys did for us in 2011.

---

**From:** Steve Runnebaum [mailto:srunnebaum@landmarksurveyinginc.com]
**Sent:** Tuesday, June 17, 2014 2:31 PM
**To:** PICARD, STEPHEN
**Subject:** NAVD 88 in 2011

Stephen,
We ran conventional levels from K 393 (magenta box) to U 300 (red box) and came up with 11.59 for the latest "true" elevation of U 300. True based on K 393 at that time. The NGS no longer publishes a current elevation on U 300, it is considered to be in a subsidence zone. They publish a little "s" at the end of the benchmark when you retrieve the datum sheet from the NGS. You can find this mark on the NGS website, but they don't publish an elevation. K 393 DOES have a published elevation on it, therefore, we ran a level loop from K 393 to U 300. If I remember, we also hit U 300 with GPS and were very close.

Let me know if there is anything else I can do for you,

***Steven M Runnebaum, PLS***
***Survey Manager***
***1513 Keubel Street***
***Harahan, La. 70123***
***Phone 504-733-3303 xt 4***
***Fax 504-734-8357***
***Cell 504-416-4435***

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**From:** BROWN Dan (AREVA) [mailto:Daniel.Brown@areva.com]
**Sent:** Monday, November 03, 2014 12:29 PM
**To:** PICARD, STEPHEN
**Subject:** RE: Waterford Survey

Thanks, I'll update the schedule.

Could you resend his contact information please, and if you could copy him on email so he is aware I have permission to ask about the work he performed for you that would be very helpful.

---

**From:** PICARD, STEPHEN [mailto:spicard@entergy.com]
**Sent:** Monday, November 03, 2014 1:28 PM
**To:** BROWN Dan (EP/PE)
**Subject:** RE: Waterford Survey

I do not know of any, but I need to look a bit more. I recommend giving Steve Runnebaum a call at Landmark Surveying. Do you still have his contact info?

On another note, sorry I have not provided comments on the hurricane, external LIP, or tsunami evaluations. We had an NRC audit for the past couple weeks and we are getting swamped with questions. I will give you comments by the end of this week.

*Stephen Picard*
**Entergy Operations, Inc.**
**WF3 Nuclear Station**
**Design Engineering - Civil**
**spicard@entergy.com**
**Office: 504-739-6376**
**Cell: 504-512-6952**
*"He who has the Son has life..." 1 Jn. 5:12*

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**From:** BROWN Dan (AREVA) [mailto:Daniel.Brown@areva.com]
**Sent:** Thursday, October 30, 2014 9:49 AM
**To:** PICARD, STEPHEN
**Subject:** FW: Waterford Survey

**EXTERNAL SENDER. DO NOT click links if sender is unknown. DO NOT provide your user ID or password.**

Stephen,

Is there any additional documentation from the Landmark Surveying work done in 2011, like a report or something along those lines?

---

**From:** BROWN Dan (EP/PE)
**Sent:** Thursday, October 30, 2014 9:26 AM
**To:** Rob Crawshaw (RCrawshaw@mckimcreed.com)
**Cc:** davidm.leone@gza.com; Chad Cox (chad.cox@gza.com)
**Subject:** Waterford Survey

Rob,

We're still struggling a bit with this so I was wondering if you had turned up any historical settlement information or have any ideas as to where we can look to rectify this.

Based on this image from a survey company that did work for the plant in 2011, it appears that there is a ~2.7 ft differential between their 2011 survey and the benchmark elevation (design vintage possibly). Unfortunately, this benchmark was outside of our survey area and was not surveyed as part of our workscope. It seems like the survey location they reference is far enough away that it wouldn't be impacted by local settlement in the plant area, which leads me to believe that the 2+ ft differential is datum based as opposed to settlement based. I also see a fairly consistent 2.2 ft difference between our new NAVD88 elevations on rooftops and the design rooftop elevations, if that adds credence to the offset.

Do you know of any external data sources that might be able to support the conclusion that the difference between the new survey and the old survey is datum based and not settlement? We also need to establish a more firm offset number which is proving difficult. If this is stuff you think we can do but it might actually require hours and $$, let me know and we can evaluate the need for a change order. If this is something that could be solved simply by getting additional documentation from the Landmark Surveying work in 2011, then I will pursue that course.

Can you call to discuss?

***Dan Brown***
Scientist IV
Environmental Health & Safety Analysis
AREVA, Inc.
7207 IBM Drive
Charlotte, NC 28262
704 805 2759 office
508 494 6330 cell
daniel.brown@AREVA.com