2.4 HYDROLOGIC ENGINEERING

2.4.1 HYDROLOGIC DESCRIPTION

2.4.1.1 Site and Facilities

Waterford 3 is located on the west (right descending) bank of the Mississippi River near River Mile 129.6 Above Head of Passes (AHP), approximately 25 miles upstream of New Orleans. The site area consists of over 3000 acres with approximately 7500 ft. of river frontage. The plant uses a once-thru Circulating Water System with the Mississippi River as a heat sink. The Component Cooling Water System serves as the ultimate heat sink and is designed to remove heat from the plant during normal operation, shutdown or emergency shutdown.

Note: During the initial phase of construction from 1975 to 1978 the plant settled approximately 9 in. Elevations at the top of the basemat which were established in the early part of this phase were used to determine the other evaluations throughout the Nuclear Plant Island Structure (NPIS). These elevations were not adjusted as the mat settled; therefore, the established elevations of the plant on design drawings are higher by approximately 9 in. than the actual elevations.

The top of the exterior walls (flood walls) of the NPIS were surveyed in 1991 to be at El. 29.27 ft. MSL. The design flood level of the NPIS is reduced to El. 29.25 ft. MSL from El. 30.0 ft. MSL, a 9 in. difference. The safety-related equipment which are housed within the NPIS are still protected from disastrous floods since the highest level the water will reach at the NPIS is El. 27.6 ft. MSL in the most severe conditions.

→(LBDCR 15-026, R309)

Several design documents state the design flood level of the NPIS or elevations throughout the NPIS without correction for the 9 in. discrepancy as stated above. In order to keep consistency among these documents and the FSAR, statements within the FSAR which state these elevations will not be decreased by 9 in. to reflect the difference between the established elevations stated on design documents and the actual elevations. (Reference: Entergy Operation Letter W3C5-91 -0138)

←(LBDCR 15-026, R309)

→(EC-29230, R305)

All safety-related components are housed in the Nuclear Plant Island Structure (NPIS) which is flood protected up to El. +30.0 ft. MSL. The NPIS is a reinforced concrete box structure with solid exterior walls. All exterior doors and penetrations below El. +30.0 ft. MSL which lead to areas containing safety--related equipment are watertight. Valves CMU-908, CMU-909, FS-201 (7FS-V1 77) and FS -202 (7F5-V608), located in the Spent Fuel Pool Cask Decontamination Area in the Fuel Handling Building, are flood barriers. During Design Basis Flooding conditions, water enters the Fuel Handling Building Train Bay and into the Spent Fuel Pool Cask Decontamination Pit. The closed valves prevent flood water from the Cask Decontamination Pit from reaching the Fuel Handling Building Sump. The plant grade around the structure varies from El. +17.5 ft. MSL on the north side to El. +14.5 MSL on the south side. Figure 2.4-1 shows the preconstruction site topography and Figure 2.4-20 shows the finished grades and drainage.

←(EC-29230, R305)

2.4.1.2 Hydrosphere

A regional map showing major hydrographic features is presented on Figure 2.4-2.

The low-lying land surrounding the site landward of the levees is part of the Mississippi River Delta Basin (Figure 2.4-2). This drainage basin is bounded by the Atchafalaya River basin to the west, the Gulf of Mexico to the south, and the Mississippi River basin to the north and east, starting at the river side of the levees. The drainage from plant site runoff flows south-westward to Lac Des Allemands, then southeastward through the Bayou des Allemands to Lake Salvador, southeastward into Little Lake and finally into Barataria Bay and the Gulf of Mexico. These lakes and waterways are used for navigation but are not a source of drinking water.

A potential cause of flooding in the Mississippi River Delta Basin is hurricane-induced surge flooding. Although the plant is approximately 60 miles from the open coast, hurricane surges have, historically, flooded large portions of the Lower Mississippi River Delta area.

The primary hydrologic feature with which the plant interacts is the Mississippi River. The plant uses the river as a sink for water heat and is protected from river flooding by levees adjacent to the plant.

The Mississippi River and its tributaries drain a total of 1,246,000 square miles, which is 41 percent of the 48 contiguous states of the United States. The River rises in northern Minnesota and flows southward for about 2,470 mi. into the Gulf of Mexico. The funnel shaped basin covers all or parts of 31 states and two Canadian provinces and is bounded on the west by the Rocky Mountains and on the east by the Appalachian Mountain Chain.[1] The lower alluvial valley of the Mississippi River is a relatively flat plain which has experienced frequent severe floods. After the disastrous flood of 1927, the Flood Control Act of May 1928 was passed for flood control in the Mississippi River Alluvial Valley. This has been modified 23 times, the latest being by the Water Resources Development Act of 1974.[2 3]

→(LBDCR 15-026, R309)

The existing comprehensive flood control and navigation plan for the Mississippi River consists of a levee system along the main stem of the river and its tributaries in the alluvial plain, reservoirs on the tributary streams, floodways to receive excess flow from the river, and channel improvements such as revetment, dikes, and dredging to increase channel capacity. Below Baton Rouge, La., 92 miles of operative revetment works are in place and a low-water navigation channel nine ft. deep and 300 ft. wide, between Cairo, Ill. and Baton Rouge, La. is maintained by dredging and dikes. Other flood control programs consist of control structures, cut-offs, pumping plants, floodwalls, and floodgates. The channel cutoff program inaugurated in the 1930's consisted of 16 cutoffs which, along with two major chutes, have reduced the river distance between Memphis, Tenn. and Baton Rouge, La. by 170 miles. This program has lowered river stages by 10 ft. at Vicksburg, Miss. at project design flood stages. Besides the flood control features, the plan provides for construction and maintenance of a navigable channel from Baton Rouge, La. to Cairo, Ills. The following are major flood control levee systems floodways and control structures near Waterford 3.

←(LBDCR 15-026, R309)

a) Levees

Below Baton Rouge, La., about 134 miles of levee are protected against river wave wash. The levee line on the west bank of the Mississippi River begins just south of Cape Girardeau, Mo., and except for gaps where tributaries join the Mississippi, extends almost to Venice, La. near the Gulf of Mexico.

b) Floodway and Diversion Structures

Security Related Information
Text Withheld Under 10 CFR 2.390

Security Related Information
Text Withheld Under 10 CFR 2.390

→(LBDCR 15-026, R309)

Table 2.4-1 lists water withdrawals for public supply along the lower Mississippi River. The ground and surface water pumpage trend for St. Charles Parish is shown in Figure 2.4-5, and according to the latest figures, groundwater preage represents less than one percent of the total water requirements.[8,9,10,11] Consequently, this community is for all practical purposes totally dependent on river water to supply its water requirements, which includes on the average 604 MGD for industrial usage.[7]

The major water supply aquifers in the St. Charles Parish area are sands in the older deltaic deposits. The major aquifers which have been identified are the “200-ft”, “400-ft”, “700-ft” and “1200 ft” sands of the New Orleans area.[12] The “400-ft” and “700-ft” sands which supply most of the pumpage have been correlated through the St. Charles Parish area. The aquifers of the New Orleans area extend westward into the Reserve-Laplace area, where the “400-ft’ sand is the principle aquifer developed. Pointbar deposits afford hydraulic connection with the Mississippi River and are sometimes used as a local source of groundwater.

In the Waterford area, the “400-ft” and “700-ft” sands in the New Orleans area are the major aquifers. Well water analysis has indicated that groundwater in the area of the site contains approximately 230 ppm chloride, over 0.3 ppm iron, and about 900 ppm dissolved solids at a temperature of 70^0F. A detailed discussion of regional and site groundwater is presented in Subsection 2.4.13.

2.4.2 FLOODS

2.4.2.1 Flood History

The major floods on the lower Mississippi River generally result from large floods on the Ohio River augmented by contributions from other major tributaries of the lower Mississippi River. The flood season on the Mississippi River is usually from the middle of December through July. The first recorded flood of the Mississippi River was described by Garciliaso de la Vega in his history of the DeSoto expedition. It occurred in 1543 and was described as severe and of prolonged duration. Fragmentary records indicate that great floods also occurred in 1782, 1785, 1796, 1809, 1815, 1823, 1844, 1849, 1858, 1862, 1867 and 1882. Major floods of recent years occurred in 1903, 1912, 1913, 1916, 1922, 1927, 1937, 1945, 1952, 1973, and 1975. For more than 200 years of record, the Mississippi has flooded, on an average, every seven years.[1] Table 2.4-2 shows the maximum confined discharges at key stations on the Mississippi River for major Mississippi River floods below St. Louis. As can be seen in the table, the largest recorded flood occurred in 1927 in the Lower Mississippi Valley below the mouth of the Arkansas River.

The flood of 1927 was the most disastrous in the history of the lower Mississippi River Valley. An area of about 26,000 square miles was inundated. The total length of levee breached in main river lines exceeded five miles.[1] Property damage amounted to about $236,000,000, which is equivalent to more than one billion 1973 dollars, 214 lives were lost and 637,000 persons were displaced. It was this flood which served as a basis for the PDF adopted by the U.S. Army Corps of Engineers for flood control and river improvement works.

←(LBDCR 15-026, R309)

The project design flood is 29 percent greater than the flood of 1927 at the latitude of Red River Landing, amounting to 3,030,000 cfs at that location. The most recent floods occurred in 1973 and 1975 on the Mississippi River.[1,2]

a) 1973 Flood

The severe 1973 spring flood in the Mississippi River basin was the most recent major flood in duration, magnitude and areal extent. The River remained above flood stages for a recorded consecutive 88 days in Vicksburg, Mississippi. The peak flow of the 1973 flood at the latitude of Red River Landing was within 3.5 percent of the 1927 flood. The River reached its highest stages since 1937. Stage hydrographs showing the 1973 floods and other significant years at Red River Landing, La.[2] are shown in Figure 2.4-6.

→(LBDCR 15-026, R309)

During the 1973 flood, numerous flood-control and multi-purpose reservoirs were in operation to reduce water levels. Table 2.4-3 shows a comparison between the 1973 observed peak discharges throughout the lower Mississippi River and what the 1973 peak discharges would have been without the existing reservoirs in the basin. The stages at Vicksburg, Mississippi were reduced about 2.5 ft. for the April crest of 1973 flood due to the reservoir operations. By May 1973, the majority of the major reservoirs experienced record elevations in flood-control and had utilized 75 percent or more of flood-control pool capacity. During the 1973 flood, the Morganza Floodway, upstream from Baton Rouge, was opened for the first time since its construction (1953) in order to divert a peak flow of 142,000 cfs through the Atchafalaya River to the Gulf of Mexico. This diversion plus a peak flow of 510,000 cfs through the Old River Control Structure amounted to about 40 percent of the flow at Vicksburg, Miss. and relieved pressure on the levees downstream. The Bonnet Carre Spillway was opened for the first time since 1950 to lower further the river stage at New Orleans by diverting a peak flow of 195,000 cfs from the river through Lake Pontchartrain to the Gulf of Mexico. With these operations of upstream reservoirs, floodways and diversion structures, the resulting crest stage and peak flow at the Carrollton gage in New Orleans were +18.47 ft. MSL and 1,257,000 cfs. The flood-crest elevations of the 1973 flood and other larger floods, in the vicinity of the Waterford site, have been tabulated (see Table 2.4-2). Figure 2.4-4 shows a comparison between the 1973 observed peak discharges throughout the lower Mississippi River and the Mississippi River Project Flood Discharge. During the 1973 flood, 12 million acres of land were inundated, 28 deaths were attributed to the flood, 50,000 people were displaced and total damages were estimated by the Corps of Engineers to be over $400 million.[2.4]

b) New Project Design Flood Flow Line

The MR&T Flood Control Project is designed to control the PDF of 3,030,000 cfs at the latitude of Old River. The project flow line and hence the project levee and floodwall grade had been established based on a computed flow line using stage discharge relationships during the floods of 1945 and 1950, and the corresponding channel and overbank conditions. The 1973 flood flow line was several feet higher than the project flow line. In developing the original PDF flow line, the possibility of a decrease in channel efficiency was considered, but no special

←(LBDCR 15-026, R309)

allowance was made for this loss. The 1973 flood analysis showed that a serious channel deterioration had taken place in the middle reach of the Lower Mississippi River since 1950. At the peak stage of the 1973 flood, the capacity of the river was about 15 percent less than the capacity of the 1950 channel. At Vicksburg, this amounts to a shift of 4.7 ft. due to channel deterioration and loop effects. The flood discharge that could safely be passed through the Atchafalaya Basin was approximately 800,000 cfs while the required discharge through the Atchafalaya Floodway during PDF conditions is 1,500,000 cfs.

For this reason, the original PDF flow lines were reevaluated and raised zero to four feet depending on location. A comparison of the 1973 adjusted and the original PDF flow lines for key stations on the lower Mississippi River is given in Table 2.4-4 (from References 2 and 3).

The annual average, maximum and minimum streamflows at Red River Landing (170 miles upstream from the Waterford site) have been tabulated from 1900 to 1976 in Table 2.4-5.

Figure 2.4-7 shows the flood frequency at Tarbert Landing, Miss. (River Mile 306.3) and at Baton Rouge, La. (River Mile 228.4) by Log-Pearson Type III Distribution method 14

2.4.2.2 Flood Design Considerations

Various hypothetical hydrologic events and combinations of hydraulic events have been used to determine the design basis for flood protection for safety-related equipment and facilities. The design bases considered and the methods used to determine them meet the recommendations of NRC Regulatory Guide 1.59 (Revision 1, 4/76). The events considered in detail are:

a) Probable Maximum Precipitation (PMP) Over the Plant Site.

→(LBDCR 15-026, R309)

The effects of the PMP on the plant site and the plant proper are presented in Subsection 2.4.2.3.

←(LBDCR 15-026, R309)

b) Levee Failure During PMF and PMH at the Mouth of Mississippi River.

The failure of the levees adjacent to the plant site was analyzed for the high water levels resulting from the PMF in the Mississippi River and the Corps of Engineers Hypo Flood - 52A in the river coincident with the PMH surge at the mouth of the river. The maximum water level resulting from the levee breach is associated with the case of the PMH surge at the mouth of the river and Hypo Flood - 52A in the river. This resulted in a maximum water level of +25.4 ft. MSL at the North Wall of the NPIS. Additional consideration of a hypothetical river stage of 30 ft. MSL resulted in a maximum effective water level of 27.6 ft. MSL. The details of these analyses are presented in Subsections 2.4.3.7 and 2.4.5.6.

c) Probable Maximum Hurricane Surge through Barataria Bay.

→(LBDCR 2008-001, R302)

The effects of a hurricane surge passing through Barataria Bay is analyzed coincident with the PMP. The maximum still water level from this analysis is computed to be +18.1 ft. MSL. The maximum effective water level from hurricane induced wind waves was computed to be +23.7 ft. MSL. The details of this analysis are presented in Subsection 2.4.7.

←(LBDCR 2008-001, R302)

2.4.2.3 Effects of Local Intense Precipitation

2.4.2.3.1 Design Criteria for Probable Maximum Precipitation (PMP)

The probable maximum precipitation (PMP) is calculated by a method which uses a combination of a physical model and several estimated meteorological parameters to yield the theoretically greatest depth of precipitation for a given duration which is physically possible over a particular area. The value is estimated by maximizing all the physical parameters responsible for extreme precipitation in previously observed heavy storms and transposing the storm orientations and trajectories to produce the greatest possible precipitation over the area of concern. Consequently, the calculated PMP is a hypothetical indication of the extreme upper limit of precipitation events.

As determined from Reference 16, the 10 square mile PMP depths for 6, 12 and 24 hours are 30.7. 34.6 and 39.4 inches respectively. The one hour rain fall increments in the critical six hour period were then arranged according to the criteria in Reference 17 such that 10 percent of the six-hour value occurred in the first hour, 12 percent in the second hour, 15 percent in the third hour, 38 percent in the fourth hour, 14 percent in the fifth hour, and 11 percent in the sixth hour.

2.4.2.3.2 Effects of PMP on the Plant Site

The plant site is located such that runoff-produced flooding from local intense precipitation will not affect the safety of Waterford 3. The site is drained externally by drainage ditches around the plant. The exterior walls of the plant are flood protected up to El +30 ft. MSL (12.5 to 15.5’ above grade) which is far above any ponding that could be expected due to a severe rainfall up to and including the PMP and assuming blocked culverts. (See note under Subsection 2.4.1.1.)

2.4.2.3.3 Effects of PMP on Roofs of Structures (Refer to Figure 2.4-8 Roof Drainage)

a) Fuel Handling Building

The Fuel Handling Building is provided with six 4 inch roof drains which exceeds the normal code design requirements by 100%. Assuming one-third of the roof drains are blocked, the remaining functional drains and storage capacity of the roof can accommodate the PMP for its duration.

b) Reactor Auxiliary Building

The Reactor Auxiliary Building is provided with six 4 inch, four 5 inch, and two 6 inch roof drains which exceeds the normal code design requirements by 100%. Assuming one-third of the drainage capacity is blocked, the remaining functional drains and storage capacity of the roofs can accommodate the PMP for its duration.

→ (DRN 99-2493)

c) Reactor Building

The Reactor Building dome and its surrounding walkway is provided with 3 six inch roof drains which exceed the normal code requirements by 50 percent. The parapet surrounding the walkway rises to a height of 21 inches. An analysis revealed that clogging of the drains was improbable, but a 33 percent blockage was considered to be conservative. Assuming one third of the walkway roof drains are blocked, the remaining functional drains and storage capacity of the walkway can accommodate the PMP with the exception of the 4th hour. 40% of the water will spill onto the Reactor Auxiliary Building roof and the remainder will spill equally into each of the Cooling Tower “A” and “B” areas.

d) Cooling Tower Areas - “A” and “B”

Both Cooling Tower areas were considered as being one large roof with regard to rainwater contribution from open areas, projected wall areas (50 percent of external walls and 100 percent of internal walls), wet cooling tower overflow, and partial spill-over from the Reactor Building parapet. The water storage capability of the Cooling Tower areas took into account the open areas (including the Fuel Handling Building air intake and exhaust plenum) less the internal walls, wet cooling tower basins, a reduction for designated storage areas, and other piping, foundations and equipment. The lowest elevation of the Fuel Handling Building (El -35 MSL) was also considered as water storage capability for the Cooling Tower areas. Water level equalization between the two areas occurs through four 4 inch pipes installed under two door sills located at each side of the Fuel Handling Building (FHB). To maintain negative pressure in the FHB, these pipes have 2 flappers installed. These flappers do not impede the flow of water into the FHB. Assuming 33 percent blockage of these penetrations the remaining openings can accommodate the necessary equalization rate. There is no safety-related equipment located at EL -35 MSL in the FHB, hence the penetrations do not impact the plant’s capability for safe shutdown and for control of radioactive releases to the environment. Also, the penetrations do not impact the plant’s Fire Protection Program (FSAR Subsection 9.5.1) in that the worst case fire with respect to penetrations is ignition and development of a combustible liquid (lubricating oil) fire. This fire is localized by floor drainage areas and curbs, and is expected to extinguish itself rapidly due to the limited amount of combustible materials available. Heat and smoke will be handled by the normal ventilation system in the FHB and will disperse to the atmosphere if there is progression to the Cooling Tower areas.

Each dry cooling tower cell, and open area adjacent to the cells, are provided with area drains. The wet cooling towers are provided with overflows at their high water level elevations, which spill onto the open areas adjacent to them. All area drains in each Cooling Tower area are interconnected by a network of drainage piping which

→(DRN 01-1108)

terminates at the area drain sump No. 1 for Cooling Tower area “A” and area drain sump No. 2 for Cooling Tower area “B”. Each drain area sump is provided with a set of motor driven sump pumps which are normally aligned to discharge to the Circulating Water system and can be aligned to the 40 Arpent Canal or to the Waste Management system. Each cooling tower area is also provided with a diesel powered sump pump. Hoses are provided that can be connected during extreme rainfall events to discharge water directly over the Nuclear Island exterior floodwall.

←(DRN 01-1108)

Loss of offsite power (LOOP) coincident with the PMP storm was considered and provisions were made to power the motor driven sump pumps from the emergency diesel generators via a manual switch. Assuming one motor driven sump pump in each cooling tower is out of service and the remaining motor driven sump pump in each cooling tower area is not operating for the first half-hour of the PMP, and that the diesel powered sump pumps are connected and started within three hours, flooding of safety related equipment will not occur.

The safety-related equipment in Cooling Tower areas “A” and “B” are Motor Control Centers 3A31 5-S and 3B315-S, and Transformers A and B. To further preclude the possibility of flooding the MCCs and the transformers, openings are provided for their respective dry cooling tower cubicles. These openings will drain water from the cubicle in the event of localized drain clogging in the dry cooling tower cells.

2.4.2.3.4 Effects of Standard Project Storm (SPS) on Cooling Tower Areas

→(DRN 01-464)

An additional analysis was performed to determine the effects of the SPS (as defined in Subsection 2.4.3.7) on the Cooling Tower areas and the safety-related equipment contained therein. Assumptions identical to those in the PMP case (Subsection 2.4.2.3.3d) were made except that no credit was taken for operation of the motor driven sump pumps.

←(DRN 01-464)

The possibility of an SSE or an OBE concurrent with the SPS was considered. The most extreme scenario would be for the SPS to commence at the same time as the seismic event. Since the probability of an SPS is not quantified, the probability of the maximum 24-hour, 100-year rainfall was used in estimating the likelihood of simultaneous occurrence of these two independent natural phenomena. The probability of the maximum 100-year rainfall is substantially higher than that of the SPS. The calculation below is therefore inherently conservative. The probability of an OBE occurring over the 40-year life of the plant is 2.6 percent. Computation of this figure is discussed in FSAR Subsection 2.5.2.7.

Therefore, the probability (P) of the simultaneous occurrence of the 100-year rainfall and OBE can be determined, using the formula of ANSI N2.12, by:

$$P = \frac{P_1 P_2 (t_1 + t_2)}{Y}$$

where,

P_1 = probability of 100-year rainfall

P_2 = probability of OBE

t_1= duration in minutes of 100-year rainfall

t_2= assumed maximum duration in minutes of ODE, and thus total pump outage

Y = number of minutes in one year

Thus

$$P = \frac{(10^{-2})(6.5x10^{-4})(1440 + 1440)}{5.256x10^{5}}$$

$$P = 3.6x10^{-8}$$

Therefore, the probability of simultaneous occurrence of an SPS and OBE is even less, and is negligible.

→(DRN 01-1108, R11-B; EC-2097, R301)

The SPS was still analyzed, however, assuming total inoperability of all motor driven pumps in order to determine the time available before levels are reached which would affect essential equipment in the Cooling Tower Areas. Assuming that the diesel powered sump pumps are started within three hours flooding of safety related equipment will not occur.

←(DRN 99-2493; 01-1108, R11-B; EC-2097, R301)

→(EC-5000082442, R301)

The diesel powered sump pumps are stored away from any non-seismic category I equipment that could fall and damage the pumps.

←(EC-5000082442, R301)

2.4.2.3.5 Effects of Ice Accumulation on Site Facilities

Ice accumulation effects at the Waterford 3 site are considered to be negligible because of the climate of the region as discussed in Section 2.3. Therefore, ice induced flooding and structural damage is not considered as design bases.

→(DRN 03-2055, R14)

2.4.3 PROBABLE MAXIMUM FLOOD (PMF) ON STREAMS AND RIVERS

←(DRN 03-2055, R14)

The PMF on the Mississippi River at Waterford 3 was determined by increasing the Corps of Engineers Project Design Flood (PDF) at the latitude of Red River Landing by 67 percent. This resulted in a peak discharge of approximately 5,000,000 cfs at that latitude. A flow of this magnitude would result in extensive overtopping of the levees above Waterford 3 and a reduction in flow at the site to levels equal to or less than those associated with the PDF. It was considered possible that a flood less severe than a PMF but more severe than the PDF might cause the greatest danger in the event of a levee failure adjacent to Waterford 3. Upon consultation with the AEC staff (now NRC), a level of El. +27.0 ft. MSL was determined to provide acceptable conservatism for the levee failure analyses.

Due to the nature of the flooding situation at the plant site, the guidance given in Appendix A of Regulatory Guide 1.59, Revision 1, 1976 was not applicable to the determination of PMF flow or water level.

2.4.3.1 Probable Maximum Precipitation (PMP)

The concept of the Probable Maximum Precipitation (PMP) is based on the continuity of flow. However, as the area and duration of a rainstorm are extended, the PMP concept cannot be applied. That is, the precipitation computed based on a sustained maximum inflow of moist air with a maximum moisture content and repeated development of maximum storm mechanisms, for several days throughout an area as large as the Lower Mississippi Basin, would be inappropriate. For a practical estimate of the POF, the Corps of Engineers and the National Weather Service (formerly U.S. Weather Bureau) adopted the hypothetical combination of precipitation storms into sequence as the basic method for estimating the PDF for the Lower Mississippi Basin.[19] From these hypothetical combinations of storms, Winter Flood No. 58A, Early Spring Flood No. 56, Late Spring Flood No. 63, and Early Summer Flood No. 52A have been estimated, both with and without upstream regulation with class EN (existing and near future) tributary reservoirs. Table 2.4-7 shows the estimated flood peak at the latitude of Red River Landing with and without upstream regulation.[20] Among these hypothetical floods, the U S Army Corps of Engineers adopted Hypoflood No. 58A, upstream regulation with class EN reservoir operating, as the PGF on the Mississippi River from Cairo, Ill, to the Head of Passes, La.[21]

→(LBDCR 15-026, R309)

The Hypoflood 58A is a winter season phenomenon consisting of a combination of the actual January 6 to 24, 1937 storm over all areas above the latitude of Red River landing with excess rainfall increased by ten percent, followed by actual January 3 to 16, 1950, storm over all areas above Cairo, Ill., and followed two days later by the February 14 to 18, 1938, storm transposed over all areas between Cairo, Ill., and the latitude of Red River Landing.

←(LBDCR 15-026, R309)

2.4.3.2 Precipitation Losses

The precipitation losses used for the determination of the PDF on the Mississippi River are discussed in Reference 20.

2.4.3.3 Runoff and Stream Course Models

The runoff and stream course models used for the computation of the PDF on the Mississippi River are also discussed in Reference 20.

2.4.3.4 Probable Maximum Flood Flow

As mentioned earlier, the PDF does not include complete storm maximization to qualify as the PMF, and if the PMF discharge is assumed to be 67 percent greater than that of the PDF, the discharge would be approximately 5,000,000 cfs at the same latitude. This discharge would result in levee overtopping and flooding of the Mississippi River Valley upstream, of the latitude of Red River Landing. The huge storage capacity provided by the valley would greatly reduce the downstream peak discharge, and the upstream levee overtopping and crevassing would also reduce the flow within the main river channel to a level expected to be less than that due to the Project Design Flood at Waterford 3. The flood control scheme at the Mississippi River downstream of Old River is shown in Figure 2.4-4.

2.4.3.5 Water Level Determinations

The water levels in the Mississippi River at Waterford 3 were estimated for the following three cases: 1) Project Design Flood; 2) Moderate Mississippi River Flood coincident with the Probable Maximum Hurricane; 3) Probable Maximum Flood.

→(DRN 01-464, R11-A; 02-123, R11-A, LBDCR 15-026, R309)

Water surface profile computation was carried out from Venice (River Mile 10.7 AHP) to Waterford 3 utilizing the HEC-II computer program[22] developed by the Hydrologic Engineering Center of the U S Army Corps of Engineers. Cross-sectional profiles from Venice to Donaldsonville (River Mile 175.5 AHP) were obtained from the Corps of Engineers at approximately one-half to two-mile intervals. The cross-sectional data is based on the 1975 Hydrographic Survey conducted by the Army Corps of Engineers. The channel capacities, from Donaldsonville to Venice, of the cross sections were compared with those of the 1961 to 1963 Hydrographic Survey and found to be a little less than the previous 1961 to 1963 capacities. For the channels, the Manning coefficients of 0.021 and 0.023 calibrated for high flows by the Corps of Engineers were used for the water level determinations from New Orleans (Mile 102.7 AHP) to Waterford 3. From Venice to New Orleans, the Manning coefficients were calibrated after several trials, using high flow conditions of April and May 1973 and April 1975 by reproducing the observed water level along the river (See Figure 2.4-9). For the overbanks, the Manning coefficient of 0.120 calibrated for high flows by the Corps of Engineers from New Orleans to Donaldsonville was used from Venice to the site. The observed water level along the river reaches and discharge data were obtained from the Post Flood Report, Flood of 1973, Vol. I[24] and Post Flood Report, Flood of 1975.[25] Figure 2.4-10 shows the PDF flow line determined by the Corps of Engineers and the computed water surface profile of the PDF using calibrated Manning coefficients up to New Orleans.

←(DRN 01-464, R11-A; 02-123, R11-A, LBDCR 15-026, R309)

Figure 2.4-9 illustrates the good comparison between the observed water surface profiles and the computed water surface profiles using adjusted Manning coefficients for several reaches.

Figure 2.4-11 shows the actual operation of the Bonnet Carre Spillway during 1973 and 1975. These curves were also used for the reproduction of observed water surface profiles of 1973 and 1975. For the projections of the flood water levels, the 1973 spillway rating curve was used, resulting in higher water levels.

The river stage of +8.5 ft. MSL at Venice for the projected flood discharge was assumed in the computation of the projected flood water levels, which the Corps of Engineers used in establishing the PDF flow line.

2.4.3.5.1 Project Design Flood

The 1973 flood flow line was several feet higher than the PDF flow line, which had been established based on a computed flow line using stage discharge relationships during the floods of 1945 and 1950 by the Corps of Engineers. Also, the 1973 flood analyses showed that serious channel deterioration had taken place in the middle of the Lower Mississippi River since 1950. At the peak stage of the 1973 flood, the capacity of the river was about 15 percent less than the capacity of the 1950 channel. For this reason, the original PDF flow lines were reevaluated and raised as much as four feet at various locations by the Corps of Engineers. A comparison of the 1973 adjusted and original PDF lines for key stations on the Lower Mississippi River is given in Table 2.4-4. However, the flow line for the PDF at Waterford 3 was not changed and is +24 ft. MSL which is the same as the original PDF flow line. Figure 2.4-12 shows the adjusted PDF flow profile and the levee design grade from the site to Venice.[26] The levee design grade in this reach is at least three feet above the PDF flow line.[27]

2.4.3.5.2 Moderate Mississippi River Flood Coincident with the Probable Maximum Hurricane

This case is discussed in Subsection 2.4.5.2.

2.4.3.5.3 Probable Maximum Flood

→(LBDCR 15-026, R309)

If the PMF of 5,000,000 cfs at the latitude of Red River Landing occurs, obviously levee overtopping and crevassing in the river reach upstream of the Waterford site would occur (See Subsection 2.4.3.4). The bankfull discharge associated with this PMF was calculated by trial and error, using the HEC-II computer program. Figure 2.4-13 indicates no overtopping would occur until the flow exceeded 1,737,000 cfs. A river stage of +8.5 ft. MSL at Venice, which the Corps of Engineers, New Orleans District used in the PDF flow line, was used to produce the levee elevation of +30 ft. MSL at the site. Also, it was assumed that the Bonnet Carre Spillway would operate at its design capacity of 250,000 cfs during the PMF. Figure 2.4-14 shows the computed river profile with bankfull discharge. As previously stated, upstream overtopping would not allow this flow to be reached. Therefore, the maximum water level considered possible at Waterford 3 is El. +27 ft. MSL, three ft. below the top of the levee.

←(LBDCR 15-026, R309)

→(EC-32952, R306)

The Mississippi River levee elevation near the Waterford site varies due to original construction and subsidence/settlement with a nominal elevation of +30 ft. MSL. This variation in elevation does not affect the design flood levels at the Waterford site because a river stage level of El. +27.0 ft. MSL is determined to provide acceptable conservatism for the levee failure analyses and the maximum effective water level at the Nuclear Plant Island Structure is based on a hypothetical river stage of 30 ft. MSL.

←(EC-32952, R306)

2.4.3.6 Coincident Wind Wave Activity

The controlling event for wind wave activity on the Mississippi River is the PMH, coincident with a moderate flood, as discussed in Subsection 2.4.5.3. In that instance, the river stage is at +28 ft. MSL and the wind velocity is 92.5 mph (10 minute duration at 30 feet above grade) when the peak of the hurricane surge reaches the site. With a PMF stage limited by upstream levee failures and diversions to +27 ft. MSL, and more moderate wind speeds, the wave climate would not be severe.

2.4.3.7 PMF-Induced Levee Failure

All safety-related equipment at Waterford 3 is protected within the NPIS, which is flood-proof to 30 ft MSL. The site itself is protected from flooding from the Mississippi River by the levee, which has a crest elevation of 30 ft MSL opposite the plant. Since this is not a seismic Category I structure, flood conditions resulting from its failure must be considered.

In the event of a flood greater than the PDF, that part of the discharge exceeding the capacity of the Mississippi main stem levees would either be stored on floodplains following levee failure or passed to the Gulf of Mexico via the Atchafalaya River basin (Figure 2.4-3). The average width of the Atchafalaya Floodway is about 15 miles and the design capacity is 1,500,000 cfs. The basin is bounded by higher ground on the west along Bayou Teche, and on the east along Bayou Lafourche. The minimum width of the Atchafalaya River basin is about 30 miles. The Waterford 3 plant is located on the edge of an extensive flat basin bounded by the Mississippi levees on the north and east and by Bayou Lafourche on the west. The high ground along Bayou Lafourche would protect the eastern basin from direct inflow of excessive discharges in the Atchafalaya River basin. In order for such a flood to enter the eastern basin and approach the plant site, the entire lowland plain from the Mississippi south of New Orleans to Bayou Teche would have to be flooded, an area approximately 100 miles in width. Since the flood would terminate at sea level in the Gulf of Mexico, there would be no backwater effect to elevate the flood. Therefore, it is concluded that flooding of the site from the Atchafalaya River basin is not possible.

→(LBDCR 15-026, R309)

Figure 2.4-1 shows the contours of the natural ground surrounding the plant to the west, east and south. Elevations vary from a maximum of 14 ft. MSL on the north and west to a minimum of 11 ft. MSL at the southeast corner of the plant area. Most of the area between the NPIS and the levee has been filled to an elevation of 17.0 or 17.5 ft. MSL. Thus, precipitation on the site or water overtopping or breaching the levee would tend to flow away from the NPIS because of the topography. In addition, water overtopping the levee directly opposite from the NPIS would be channeled away to the west or the east by the raised embankment for Highway 18 which adjoins the levee. It is estimated that the surge from a slow-moving PMH, crossing over the low-lying marshlands from the direction of Barataria Bay, could exceed the plant grade of 17.5 ft. MSL by 0.6 ft. for a brief period (Subsection 2.4.5.2). However, the coincidence of this event with a river flood greater than the PDF is not considered reasonably possible.

←(LBDCR 15-026, R309)

The only conditions conducive to flooding of the site itself are heavy precipitation and failure of the levee. Figure 2.4-20 shows the area between the NPIS and the levee, including the location of the Administration Building, the elevated relocation of Highway 18, and the raised plant grade area. This area is drained by a ditch along the toe of the highway embankment, and by a series of catch-basins and storm sewers leading to ditches on the east and west sides of the NPIS. The drainage system is designed for a maximum rainfall intensity of 8.25 in. per hour.

In determining the site conditions prevailing prior to levee failures, it is assumed that the Standard Project Storm (SPS) occurs coincidentally with the flood greater than the PDF. Hourly maximum point precipitation values for return periods of up to 100 years are given by Reference 28. At the Waterford 3 site, the 100 year extreme one-hour rainfall is 4.5 in. Reference 29 describes the procedure for determining an SPS from a PMP. As determined from Reference 16 and discussed in Subsection 2.4.2.3, the six-hour, ten square mile PMP is 30.7 in. For the site area, the SPS is taken as 60 percent of the PMP[29], or 18.4 in. In the critical hour of the six-hour period, 38 percent of the rainfall[29] or 7.0 in. occurs. By comparison with the 100-year one hour point precipitation value from Reference 28, 7.0 in. is a reasonable value for a one-hour SPS. Thus, the yard drainage criterion of 8.25 in. per hour is sufficient to pass the SPS, and there will be no standing water present on the site prior to levee failure.

The levee is assumed to fail completely and instantaneously, and the length of the breach is sufficiently great that spreading effects are negligible at the center of the flow, in which the NPIS is located. Instantaneous levee failure is hypothesized to occur as a result of either piping or toe erosion which undermines the embankment. The former requires the presence of sand lenses or other permeable strata beneath the levee: the latter occurs in pointbar deposits.[31] Since neither of these conditions occurs at the site, the hypothesis of instantaneous failure is very conservative. No credit has been taken for the presence of the elevated roadway parallel to the levee. Although this is an engineered structure nearly equal in height and cross section to the levee, it is conservatively assumed to fail along with the levee.

→(LBDCR 15-026, R309)

A river stage of 27.0 ft. MSL has previously been established as a reasonably conservative design basis for levee failure. (Amendment No. 9 to the PSAR, January 1972, Question 2.22.3). In this event, the effective head which determines the velocity of the flood is taken as the difference between the river stage, 27 ft MSL, and the lowest grade present on the landward side of the levee, 14 ft, MSL.

There is an analogy between the sudden levee crevasse problem and the dam break problem where the theoretical studies[32~33] and the experimental study[34] summarized on Figure 2.4-21 as nondimensional curves.

The water profile from the theoretical study[32], neglecting the bed resistance under the sudden failure condition, can be expressed as:

←(LBDCR 15-026, R309)

$$\frac{X}{t\sqrt{gy_o}} = 2 - \frac{v_o}{\sqrt{gy_o}} - 3\sqrt{y/y_o} \qquad (13)$$

→(DRN 06-869, R15)
where X is the distance from the levee, t is the time, y_0 is the water depth of the river above the breached level (= 13 ft.) v_0 is velocity, in the river perpendicular to the levee (~0), and y is the water depth in front of the levee at a distance X. The velocity along the water profile can be computed by the formula from Reference 32:
←(DRN 06-869, R15)

$$V = 2C_o - 2C + v_o \tag{14}$$

where:

→(EC-30923, R305)

$$C_o = \sqrt{gy_o}\text{ , and } C = \sqrt{gy} \tag{15}$$

←(EC-30923, R305)

Equation (14) is valid for zero bed slope and zero bed resistance. (It should be noted that Equations (13) and (14) are somewhat different from those in the reference because the coordinate system is reversed.) It should be recognized that there is no surge or abrupt wave front. The water would behave as a long wave and its dynamic effect on a structure can be evaluated by its velocity head.

From Equation (14), the velocity of the leading edge would be approximately 41 ft. per second (for y_0 = 13 ft.) and water depth is infinitesimal, from which it is evident that the assumption of zero resistance is unrealistic. Consequently, the water profile from the physical experiment where the bed resistance was considered should be used instead.
→(LBDCR 15-026, R309)
The profile on Figure 2.4-21, with resistance appears to have a sharp front in the leading edge. In fact, it is not so. The curves in Figure 2.4-21 are presented in the dimensionless form. When expressed in the dimensional form where X = 600 ft., the length of the leading edge from y/y_0 = 0.2 to y/y_0 = 0 is about 42 ft. (600 x 0.07). With the water profile changing from 2.6 ft. to 0 ft. in a distance of 42 ft., the leading edge should be considered as a long wave. Therefore, there will not be a surge formation, and the dynamic effect is that due to the velocity head alone ($v^2/2g$).

Assuming that Equation (14) is applicable from y/y_0 = 0.5 to y/y_0 = 0.2 on Figure 2.4-21, the static and the dynamic heads can be computed and are given in Table 2.4-8. The highest water level that would be expected on the wall of the NPIS due to flood caused by PMF induced levee failure is at EL 24.6 ft. MSL 5.4 ft. lower than the design level. For water level caused by PMH surge through Barataria Bay see Subsection 2.4.5.2.C.

The event of a PMH surge propagating up the Mississippi coincident with Hypo Flood 52-A and with a sudden, complete levee failure, is discussed in Subsection 2.4.5.6. The peak static plus dynamic water level against the NPIS increases to 25.4 ft. MSL and the calculation is also shown in Table 2.4-8 as Case 2.

The elevation of the crest of the Mississippi River levee at the Waterford 3 site, 30.0 ft. MSL, represents the greatest possible river stage. Any increase in flow at this stage would overtop the levees, and be stored on floodplains which are many miles in width and drain to the Gulf of Mexico. Whether a stage of 30 ft. MSL is actually possible, given a flood greater than the PDF, would depend on the modes of failure of the system of levees and floodways on the Mississippi and its tributaries. The stage at the site associated with the PDF is 24 ft. MSL. At this point, floodways and levees upstream from the site would be operating at maximum capacity. Any further increase in flood flow would begin to cause
←(LBDCR 15-026, R309)

failure of levees and flood control structures at upstream locations, diverting flow from the Mississippi main stem and reducing the crest of the flood.

However, due to the difficulty of establishing the expectation of flow diversion, and to the possibility of future channel changes, results of analyses using the upper-limit stage of 30.0 ft. MSL are presented. It should be noted that this stage is the result only of flood discharge, not of wind wave action superimposed on a lower flood stage. Although wave action induced by the PMB coincident with Hypo Flood 52A could produce momentary water levels exceeding 30.0 ft. MSL (see Subsection 2.4.5.3), the stillwater level of 28 ft. MSL under these conditions would determine the rate of flow through a levee breach. Thus, the flood stage of 30.0 ft. MSL is the critical condition.

The previous analysis is repeated for this increased assumed river stage.

Table 2.4-9 gives values of velocity, velocity head, and total static plus dynamic head for various depths. The leading edge of the flood wave, represented as $y/y_0 = 0.2$, produces the maximum velocity and thus the maximum elevation, 27.6 ft. MSL. The head is equal to the vertical elevation between the crest of the levee, 30 ft. MSL, and the plant grade, 17.5 ft. MSL. Although there is a slight uphill grade from the levee to the NPIS, it is conservatively approximated as flat. This is a slight conservative modification of the procedure used for the 27 ft. MSL stage, in which the grade level was taken as +14 ft. MSL. Use of an elevation less than 17.5 ft. MSL would yield less conservative results.

Since the NPIS is flood-protected to elevation 30 ft. MSL (see note under Subsection 2.4.1.1), a flood resulting from instantaneous levee failure with the Mississippi at the maximum possible stage of 30.0 ft. MSL would not endanger any safety-related facilities. The maximum effective water level against the north wall of the NPIS would be 27.6 ft. MSL.

2.4.3.8 Scour Associated With Levee Failure

→(LBDCR 15-026, R309)

In the event of a levee failure with the river at the assumed stage of El 27 ft. MSL, the water flowing out of the breached section will initially tend to flow at critical depth and spread. The water is assumed to erode the surface material (clay) until the bottom shear stress exerted by the flowing water is equal to or less than the critical shear stress of the material (Tc).

←(LBDCR 15-026, R309)

In separate studies, Dunn (1959) and Flaxman (1963)[35], as well as other researchers, have attempted to correlate the critical shear stress of a cohesive material with the more readily available mechanical properties of that material. Dunn correlated critical shear stress with Plasticity Index and vane shear strength, whereas Flaxman correlated critical shear stress with unconfined compressive strength. Using these studies and the average values of the recent alluvial surface material (grade to El -40 ft. MSL) as presented in Subsection 2.5.3.2. an average critical shear stress of 1.1 lbs/sq ft was determined. The upper material was assigned a critical shear stress of 0.1 lbs/sq ft. The material from El -40 to El -30 was assigned a critical shear stress of 1.0 lbs/ft^2.

As the initial breach begins to erode, it is assumed that a rectangular channel will be formed such that the maximum shear stress on the sides and bottom of the channel will be equal. Relationships have been developed by the U.S. Bureau of Reclamation and presented by

→ (DRN 02-123, R11-A)
Chow[33], for maximum shear stress on the sides and bottom of rectangular and trapezoidal channels. From these relationships, it may be seen that a channel width to depth ratio of 1.75 to 1.0 has an equal shear stress of approximately 0.62 γyS on both sides and bottom, where:
←(DRN 02-123, R11-A)
→(DRN 01-464, R11-A)

γ is the unit weight of water,
y is the depth of flow, and
S is the friction slope.

←(DRN 01-464, R11-A)

Replacing S with $\frac{n^2 v^2}{2.21y\,4/3}$ and assuming n = 0.02, the depth of erosion as a

function of velocity may be determined as $\left(\frac{0.77\gamma n^2 y^3}{T_c}\right)^3$ at the inital

opening. Because large flows through the breach opening will reduce the river stage, a trial and error solution is necessary to determine the proper velocity and corresponding depth of erosion. Various velocities were assumed and the depth of erosion and spreading of flow was calculated. The total friction loss was then computed and the net head differential was checked against the head differential necessary to provide the assumed initial velocity through the breach. A velocity of 14 fps was determined in this manner. This velocity resulted in a depth of erosion of 53 ft below El 22 ft (swl of flow) and a width of 93 ft. The maximum extent of erosion is therefore down to El -31 ft. MSL at the hypothesized levee breach opening.

→(LBDCR 15-026, R309)
The degree of flow spreading was calculated using the relationship
←(LBDCR 15-026, R309)

$$z/b_1 = 1/2\left(\frac{x}{b_1 F_1}\right)^{1\ 5} + 1/2$$ from reference 33

where:

z is the half width of flow at a distance x
b_1 is the initial width of flow
F_1 is the initial Froude Number

Because the Froude Number varies with the width of the channel and the depth of erosion, the equation was applied initially to short reaches of the expanding flow.

Using this step procedure and calculating a depth of erosion by trial and error at the ends of each reach in the expanding channel, a water surface and scour profile was computed from the levee to the marsh. The maximum depth of flow adjacent to the plant was calculated to be 9.5 ft which represents a scour down to El + 12.5 ft. MSL.

In addition to the head loss due to transportation of the eroded material, velocity and erosion would also be reduced by the gradual deposition of this material downstream of the levee breach as the flow spread. This would tend to increase the grade of the ground surface, decreasing the head differential available to push flow through the breach opening. Although these factors could not be taken into account analytically, they are considered to be significant and the analysis is considered to be conservative.

2.4.4 POTENTIAL DAM FAILURES, SEISMICALLY INDUCED

→(LBDCR 15-026, R309)

The nearest flood control reservoir to the site on the Mississippi River Basin is the Grenada Reservoir on the Yalobusha River[36] in northern Mississippi. This reservoir, which impounds approximately 1.3 million acre-ft. of water at maximum capacity, is over 550 river miles from the site. Other flood control reservoirs on the Yazoo Basin further upstream from the site include Enid Reservoir, Sardis Reservoir and Arkabulta Reservoir. These reservoirs are capable of impounding, at maximum capacity, approximately 660,000 acre-ft., 1.6 million acre-ft., and 525,000 acre-ft., respectively (see Reference 37).

Although the combined storage of those reservoirs is considerable, the stream distance and resulting channel storage between the reservoirs and the plant site is considered to be great enough to attenuate any flood wave resulting from the failure of any of these reservoirs to a level below that resulting from the PMF, or a PMH at the mouth of the Mississippi River. The reservoirs are not in tandem: therefore, the combined failure of all the reservoirs is not considered to be reasonable.

Nearer to the site are the Old River Control Structure at river mile 315 AHP, the Morganza Floodway at river mile 280 AHP, and the Bonnet Carre Floodway at river mile 128 AHP, just downstream of the plant site. The failure of any or all of these structures would result in a lowering of the water level at the site, not an increase.

In conclusion, the seismic failure of upstream dams does not present a threat to the site and is not analyzed further in this subsection. The failure of the river levees adjacent to the plant due to wave overtopping or other causes is analyzed in Subsections 2.4.3.7 and 2.4.5.6.

2.4.5 PROBABLE MAXIMUM SURGE AND SEICHE FLOODING

2.4.5.1 Probable Maximum Winds and Associated Meteorological Parameters

The Mississippi Delta region of Louisiana is prone to high winds and flooding associated with hurricanes, which will clearly be the agents most likely to induce surge flooding. Although the site is 129 miles above Head of Passes, and approximately 60 miles north of the open waters of the Gulf of Mexico, there exist possible pathways by which a severe hurricane surge could approach the site or aggravate a preexisting river flood. Therefore a Probable Maximum Hurricane is hypothesized. According to the definition of the National Weather Service given in Reference 38:

> "A hypothetical hurricane having that combination of characteristics which will make it the most severe that can probably occur in the region involved. The hurricane should approach the point under study along a critical path and at optimum rate of movement."

A total of 86 tropical cyclones have approached the coast of Louisiana in the period from 1900 to 1975. Of these, 26 have been hurricanes having central pressure indices below 29.00 in. of mercury[38], and 18 of these caused major damage in Louisiana[39]. Surges from some of the major hurricanes have approached the site, to within a few miles, in three directions: up the Mississippi River from Head of Passes, across the low-lying wetlands to the south from

←(LBDCR 15-026, R309)

the open Gulf, and through Lake Pontchartrain from Mississippi and Chandeleur Sounds.

The extent of flooding and tide gage records for these hurricanes have been documented in Reference 39. Pertinent aspects of storms which threatened the Waterford site vicinity are described below:

<u>1909, September 20</u> This hurricane, with 80 mph winds and of great extent, had the results typical of a storm traveling north-northwest and passing over shore just west of the mouths of the Mississippi. The maximum recorded tide at the coast was 15 ft. at Sea Breeze, and most of the low-lying wetlands of the delta region were flooded to elevations of 4-5 ft. MSL. Slightly higher land, such as the natural levees along Bayou Lafourche, and a band 5-10 miles wide along the Mississippi, remained above water. The surge entered Lake Pontchartrain and inundated the wetlands between the lake and the levee on the east bank of the river. The maximum water level in the lake was +8 ft. MSL.
→(LBDCR 15-026, R309)
<u>1915, September 29</u> This hurricane had a path very similar to that of the hurricane of 1909, but greater intensity (94 mph at Burrwood). The flooding pattern was similar, except that a particularly high surge occurred on Lake Pontchartrain, a stage of +13 ft. MSL being recorded at Frenier. The maximum surge on the open coast was +9 to +10 ft. MSL at Grand Isle; +55 ft. MSL was recorded on Little Lake.

<u>1947, September 19</u> Traveling west-northwest toward the converging coasts of Louisiana and Mississippi, this very intense (98 mph) hurricane produced a high surge on the order of +11 ft. MSL in Lake Borgne, but little on the southern coasts. Although large areas along Lake Pontchartrain were flooded, the water levels in the western half of the lake were only +3 to +55 ft. MSL.

<u>1956, September 23 (Flossy</u>) Hurricane Flossy crossed the lower delta in a northeasterly direction, flooding most of the low-lying delta wetlands. Maximum stages were +12.1 ft. MSL in Breton Sound and +73 ft. MSL at the west end of Lake Pontchartrain.

<u>1961, September 11 (Carla</u>) Carla passed far south of Louisiana, and most of the wetlands were again flooded. Maximum elevations were typically +5 ft. MSL.

<u>1965, September 9 (Betsy</u>) The most severe of historical hurricanes in terms of flooding, Betsy traveled northwest across Grand Isle with 105 mph winds. Tides were +8.8 ft. MSL there, +15.7 ft. MSL in Breton Sound, and +12.1 ft. MSL at the west end of Lake Pontchartrain. Only narrow strips of land along the Mississippi and Bayou Lafourche were spared from flooding, with water 2.8 ft. above sea level 13 miles south of Waterford. Stages on the Mississippi River were +12.4 ft. MSL at New Orleans and +13.1 ft. MSL at Bonnet Carre.

<u>1969, August 17 (Camille</u>) This hurricane had the greatest intensity of any documented hurricane in the mid-Gulf zone. Camille passed just east of the Mississippi delta and hit the Mississippi coast with winds estimated at 160 mph (gusting to 200 mph), and tides as great as +22.6 ft. MSL. The maximum stage in Louisiana was +15.9 ft. MSL in the lower delta: little flooding occurred west of the river mouths. The Mississippi River in New Orleans peaked at +11.5 ft. MSL.
←(LBDCR 15-026, R309)

Basic parameters to define the PMH were selected from the National Weather Service memorandum HUR-7-9.[38] Those values pertinent to Zone B (Mid-Gulf) and to Mile 660 (New Orleans) were used. These parameters, and additional information required, are given below.

→(LBDCR 15-026, R309)

a) PMH Path

Of the three possible approaches of a surge to the site, the upriver path will produce the maximum still-water level at the site. The path through Lake Pontchartrain would require overtopping of levees on both sides of the river before a surge could reach the site; this is not considered reasonably possible. The direct overland path from the south will be discussed later in this section.

In order to cause a maximum surge to propagate up the Mississippi, the PMH should approach Head of Passes along a line perpendicular to the general trend of the bottom contours. The eye should make landfall west of Head of Passes by a distance equal to the radius of maximum winds. This path is shown in Figure 2.4-22.

b) CPI (P_0)

The Central Pressure Index (CPI), the lowest pressure in the eye of the PMH, is given as 26.90 in. of mercury in Reference 38, Table 1.

c) Asymptotic Pressure (P_n)

The asymptotic pressure P_n, the ambient pressure at the outer edge of the hurricane circulation, is given as 31.26 in. Hg in Figure 6 of Reference 38.

d) Radius of Maximum Winds (R)

The distance from the eye to the point of maximum wind velocity is called, R, Representative small, medium, and large values from Table 1 of Reference 38 are seven, 14, and 30 nautical miles respectively.

←(LBDCR 15-026, R309)

e) Forward Translational Speed (T)

Slow, medium, and high forward speeds of the PMH were given as 4, 11, and 28 knots respectively.[38]

f) Maximum Wind Speed (V_x)

For different combinations of radius of maximum winds and translational speeds, maximum wind speeds were calculated using the following equations from Reference 38.

$$V_{gx} = K\left(P_n - P_o\right)^{1/2} - R\left(0.575 f\right)$$

$$V_x = 0.885 V_{gx} + 0.5T$$

where:

V_{gx} = maximum gradient wind speed in mph

P_o = central pressure in in. Hg

P_n = peripheral pressure in in. Hg.

K = an empirical constant

R = radius of maximum winds in nautical miles

V_x = wind speed at a height of 30 ft. in mph

T = forward speed of storm in mph

f = Coriolis parameter

Table 2.4-10 shows the values of Vx, corresponding to the different combinations of R and T, which were used in the surge computations.

2.4.5.2 Surge and Seiche Water Levels

The open coast surge hydrograph was calculated according to the bathystrophic storm tide theory, as developed by Marinos and Woodward[40] and programmed by the Coastal Engineering Research Center in Reference 41. Basically, the theory describes the phenomenon of storm tide rise along the coast caused by: (1) the direct wind stress acting on the surface of the water and (2) the additional effect created by the earth's rotation on the along-shore current known as the Coriolis and Bathystrophic effect. It is an unsteady quasi two-dimensional mathematical model and is different from the usual steady, one-dimensional approach which accounts for only the rise caused by the wind stress acting perpendicular to the coastline.

→(LBDCR 15-026, R309)

The basic assumptions imposed on the theory are that (1) there is no sustained mass transport toward shore across the bottom contours; (2) the inertial effect is negligible with respect to the Coriolis effect; (3) the change in sea surface along the shore is insignificant; and (4) the divergence of the velocity field does not bring about significant changes in the height of the water surface.

←(LBDCR 15-026, R309)

Based on the preceding assumptions, the integration of the equations of motion over the depth gives the following governing equations:

$$\frac{\partial F}{\partial t} y = kUU_y - K_b F_y^2 h^{-7/3}$$

$$\frac{gh\partial S}{\partial X} = kUU_x + fF_y$$

F_y = mass flux normal to the depth contours

k = the dimensionless wind stress coefficient

U = the actual wind speed

U_x, U_y = the x and y components of the wind speed

K_b = the sea bed friction coefficient

S = the setup of water elevation due to hurricane

h = the water depth including “5”

g = the gravitational acceleration

f = the Coriolis parameter

The bottom friction coefficient was conservatively taken to be 0.0001. The wind stress coefficient is of the form:

$$k = K_1 + K_2 (1\text{-}16/U)^2$$

when U is in mph.

Marinos and Woodward recommended values of $K_1 = 1.2 \times 10^{-6}$ and

$K_2 = 1.8 \times 10^{-6}$.[40]

→(LBDCR 15-026, R309)

Additional information required for surge modeling is the ambient surface elevation, a combination of astronomical tide level and an initial rise observed to occur prior to hurricanes. The maximum value of the initial rise for Louisiana (from Reference 42) is given as 2.0 ft. Present requirements specify use of the 10 percent exceedance spring tide, which has been estimated at +2.0 ft. MSL (see Reference 43). Thus the total ambient surface elevation is +4.0 ft. MSL. The hurricane wind fields are given as input in the form of dimensionless digitized profiles, developed for different values of R (the radius of maximum wind) by the National Weather Service (see Reference 38).

←(LBDCR 15-026, R309)

The final information required is the bottom profile along a line perpendicular to the general trend of the isobaths, extending to a depth of 600 ft. The bottom rises steeply from this depth to the entrance of South Pass, over a distance of nine nautical miles. An additional five miles of water shallower than 40 ft., on either side of the pass itself, is considered to contribute to the surge before it reaches the effective open coast. The depths and corresponding distances are given in Table 2.4-11.

a) Open Coast Surge Hydrograph

Surge hydrographs were computed for the nine cases listed in Table 2.4-10. Cases 3 and 9, combining the maximum radius with either the slowest or fastest forward speed.

produced the greatest peak surge, 16.1 ft. MSL. Table 2.4-10a compares the components of the surge for the two cases. However, the slow-moving PMH is critical with respect to food levels in the Mississippi River because of the much greater duration of the backwater effect at the mouth.

The hydrograph shown in Figure 2.4-23 represents the water surface elevation at the end of the delta land, eight miles below the Head of Passes, and 19 miles downstream of Venice.

b) Peak Surge at Waterford 3

→(LBDCR 15-026, R309)

To establish the peak surge of the Mississippi River at the Waterford 3 Site, the PMH is assumed to coincide with a moderate river flood. Hypo Flood 52A, an early summer design flood discharging 1,250,000 cfs south of Red River Landing, was chosen for this purpose (see Reference 20). The coincidence of these two events is not unreasonable, since two major hurricanes in the period of record have occurred in June, and four in July. The peak of Hypo Flood 52A occurs early in July.

Since it is not possible to compute the surge height at Venice or Head of Passes, because of complex topography, the open coast hydrograph is transposed to Venice. Friction losses over the lower delta are not expected to be significant, because of the low elevation and partial submergence of the land, and because water will flow laterally across the narrow delta land from adjacent open waters subject to surge.

The computed total storm surge of +16.1 ft. MSL at Venice is routed to the Waterford 3 site with the upstream flood discharge of 1,250,000 cfs utilizing the HEC-II computer program with adjusted Manning coefficient (from Venice to New Orleans), and the Corps of Engineers' Manning coefficient (from New Orleans to the site), using 1973-1975 cross-sectional profiles. The resulting river stage at the site without the local wind effect is found to be +25.2 ft. MSL. The river stage without the effect of the PMH is +21.5 ft. MSL.

As discussed in Subsection 2.4.3.5.1, the 1973 flood clearly demonstrated that the Project Design Flood could occur as a result of a succession of moderate to large storms, which would give a stage discharge loop curve similar to the 1973 flood. As a result, at the project flood discharge, the stage increases for loop-effect were added to the stage increases for channel deterioration (see Table 4 of Reference 2). Since the scour and deposit of the Mississippi River is beyond the scope of Reference 2, three ft. of stage increase was added to the computer value, assuming that similar future channel deterioration and loop effect could occur between the site and Venice. Thus the river stages with and without the effect of the PMH were determined as 28 ft. MSL and 24 ft. MSL respectively.

←(LBDCR 15-026, R309)

The net effect of the PMH on the plant site for an upstream discharge of 1,250,000 cfs is to increase the stage by four ft. From Figure 2.4-24 it can also be observed that the net effect of the open coast hurricane surge is gradually diminished as it travels upstream. It should be mentioned that the results shown on Figure 2.4-24 do not consider the effect of local wind on the surge propagation. The hurricane wind field is cyclonic and moving, and since the river is quite meandering, the wind may

→(LBDCR 15-026, R309)

have positive effects at one segment and negative effects at the other. Therefore, it is reasonable to assume that the net effect on the surge propagation due to local wind is zero.

←(LBDCR 15-026, R309)

c) Open Coast Surge: Direct Path

The land south of the Waterford site is mostly low-lying marshland, much of which is below +3 ft. MSL. The marshes are crossed by roads and a railroad at somewhat higher elevations, about eight miles southeast of the site. Historical hurricanes have caused increases on the water levels of the lakes and bayous, and have flooded parts of the marshland in this area.[39] Thus it is desirable to determine whether PMH-induced flooding could threaten Waterford 3 from the south.

An open coast surge hydrograph has been computed, according to the methods previously discussed, for a PMH whose region of maximum winds approaches Barataria Bay in a direction perpendicular to the coast. (The bottom profile is given in Table 2.4-11; the track on Figure 2.4-22.) The maximum surge is found to be +17.2 ft. MSL for the slow speed of four knots and the large radius of 29 nautical miles.

In computing the water surface profiles (shown on Figure 2.4-21) as the surge travels inland, the area southeast of Waterford 3 can be approximated as a rectangular basin. It is bounded on the west by the natural levees along Bayou LaFourche, and on the east and north by the Mississippi River levees. It may be conservatively assumed (although reasonably, for a large-radius hurricane) that the surge height is equal at all points along the coast, and therefore that the water does not flow out of the basin to the west.

A computer program based on the two-dimensional model of Reid and Bodine[44] has been developed to numerically solve the one-dimensional equations of motion for the rectangular basin. The effects of wind stress and bottom friction, the ground elevations, and the PMP are incorporated. Expressed in finite difference form, the equations are as follows:

$$U(i+1.n+1) = \frac{U(i+1.n) + \dfrac{g\Delta t}{2\Delta S}[D(i+1.n) + D(i.n)][H(i.n) - H(i+1.n)] + x(i-1.n)\Delta t}{1 + f_b \Delta t \; 4U(i+1.n)[D(i.n) + D(i+1.n)]^2}$$

$$H(i.n+1) = H(i.n) + \frac{\Delta t}{\Delta S}[u(i.n+1) - u(i+1.n+1)] + R(i.n)\Delta t$$

where:

i counts steps in the x-direction (inland)

n counts steps in time

Δt and ΔS are the finite differences in time and distance respectively

U is mass transport in the x-direction in ft.^{3}/sec.-ft.

H is the water surface elevation in ft. MSL

D is depth, H-Z

Z is bottom or ground elevation in ft. MSL (positive upward)

f_b is the bottom friction factor

R is rainfall in ft./sec.

$X = W^2 \cos\theta \, [1.1 \times 10^{-6} + 2.5 \times 10^{-6} (1 - 16/W)^2]$

W is the 30-ft. sustained wind velocity in mph

Θ is the angle between the x-axis and wind direction

→(LBDCR 15-026, R309)

Reid and Bodine describe methods of incorporating such features as weirs and sills in the computational scheme. In the present computations, these methods are used to model the shoreline as a sill at 0 ft. MSL (discharge coefficient Cs = 0.4), and Route 631 as a broad-crested weir at +9. ft. MSL (Cs = 0.46). The algorithm parametrically varies the value of the denominator of Equation (1), and makes the distinction between free-flowing and submerged weirs.

Wind velocities are determined from Figure 12 and Tables 2 and 3 of Reference 38. Ten mile steps in the x-direction are considered to give sufficient resolution of the water surface. Stability considerations require a 15-minute time step for this distance, and computations proceed in steps from offshore to the inland limit of the basin at each time step. Calibration studies to determine the correct bottom friction factor (rb) have been performed using maximum surge heights observed during Hurricane Betsy of 1965 and the hurricane of September 29, 1915, as reported in Reference 39. Figure 2.4-25 shows the maximum water levels computed for Betsy, the hurricane of 1915, and the PMH. A reasonable fit to the historical data is obtained for a value of ~b = 0.01. This is high compared to Reid and Bodines optimum value of 0.0025, but reflects the fact that the water must travel over land. The calibration by Reid and Bodine was performed for astronomical tides in Galveston bay, which would be influenced by the much smoother bay bottom.

The predicted height of the PMH flood at the site is +18.1 ft. MSL, including PMP. It is observed that both the computed and historical flood profiles, for Betsy and the hurricane of 1915, slope downward from shore to land. Compared to these storms of moderate forward speed (Betsy, 17 knots; 1915, 10 knots), the slow-moving (four knots) PMH has time to induce a much greater shoreward transport. The beginning of this effect can be seen in that the area beneath the profile of the 1915 storm is greater than that of the faster-moving Betsy. The extreme is reached in the PMH, when a great mass of water has moved over land, some 10 days after the waters began to rise, is driven uphill by the sustained winds. The water level is above plant grade, +14.5 MSL on the south side, for 27 hrs. in the event of coincident PMH and PMP. This assumes that the entire PMP for a 1000-square mile, 48 hr. period (32.6 in.) occurs prior to the time of peak surge at the site.[16]

←(LBDCR 15-026, R309)

2.4.5.3 Wave Action

To determine the highest water level to be expected at Waterford 3 due to the PMH and the upstream flood, the wind setup and wave runup due to the coincident local hurricane wind should be included.

→(LBDCR 15-026, R309)

It takes about four hours for the peak PMH surge to travel from the coast to the plant site. During this time period, the eye of the Probable Maximum Hurricane will travel 16 nautical miles (with four knots forward speed) and will still be over water (see Figure 2.4-22). The reduction in the maximum wind speed may, therefore, not be appreciable, and will still be 138 knots. The distance from the hurricane eye to the plant site at the time that the peak of surge reaches Waterford 3 is about 70 nautical miles, and the actual distance to radius of ratio (r/R) is equal to 2.3. The surface wind speed is computed to be 105 mph maximum winds at the site.[38] Since the hurricane center is very close to shore, some portions of the wind field will be overland. The surface wind speed in these portions of wind field will be reduced. From Table 3 of Reference 38, the adjustment factor is linearly interpolated to be 0.88. The expected 10-minute average surface wind speed at the site is therefore calculated as 92.5 mph or 136 fps.

←(LBDCR 15-026, R309)

→(DRN 01-464, R11-A)

Since the probable maximum hurricane track is hypothesized, it can be oriented such that it produces the most adverse local wind direction at Waterford 3 site. The effective wind direction which causes maximum wind setup and wave runup on the levee is that normal to the levee. Thus it is adequate to consider the wind setup and the wave runup caused by the local wind blowing normal to the levee, or a northeasterly wind. The effective wind fetch (Fe) is approximately 4000 ft. and the average water depth is estimated as 75 ft. Then from equation (5.11) of Reference 45, the wind setup is computed as 0.1 ft., which is negligible. Significant wave characteristics generated by the wind are computed as follows:

←(DRN 01-464, R11-A)

$$\frac{gF_e}{U^2} = \frac{32.2 \times 4000}{(136)^2} = 7.0$$

$$\frac{gd}{U^2} = \frac{32.2 \times 75}{(136)^2} = 0.13$$

where:

g = gravitational acceleration

F_e = effective fetch

U = wind velocity. ft/sec

d = depth of water

From Equation 3-21 Reference 46 we obtain

$$\frac{gH_S}{U^2} = 8.0 \times 10^{-3}$$

or

H_s = 4.6 ft. (significant wave height)

From Equation 3-22 of the same reference, we have

$$\frac{gT_S}{2\pi U} = 0.15$$

or Ts = 4.0 sec (significant wave period)

The wavelength L = 5.12Ts2 = 82 ft.

The maximum wave height = 1.67H_5 = 7.7 ft.

The wave runup for an embankment slope of 1.4 is calculated, according to Reference 46, Figure 7-12, as 5.1 ft. for the significant waves, and 5.9 ft. for maximum waves. The maximum water level stage, with the inclusion of wave runup, would cause the levee to be overtopped.

→(LBDCR 15-026, R309)

In the case of a hurricane surge approaching Waterford 3 over the marshland to the south, the maximum still-water level has been found to be +18.1 ft. MSL. Waves will be generated by southeasterly to southwesterly winds, since the eye of the PMH is even with the site when water first reaches the plant grade, and is well past it by the time of peak surge. Effective wind velocities at these times can be determined from Figure 12 (dimensionless wind profiles) and Tables 2 and 3 (filling and overland adjustment factors) of Reference 38. At the time of the arrival of the leading edge of the surge, the wind velocity is 92 mph. This is the maximum wind of the hurricane at this time, and has therefore been assumed to be oriented parallel to the long axis of the basin. The fetch is taken as the distance to the coast, 58 miles, although the wind velocity is less at the southern end. When the surge peaks at the site, the velocity has dropped to 73 mph, and the direction has shifted about 45 degrees, reducing the fetch to 26 miles.

←(LBDCR 15-026, R309)

Dimensionless relationships for wave generation in shallow or transitional waters are given in Reference 46. When the surge first reaches plant grade, the significant wave height and period are computed to be 5.0 ft. and 3.6 sec. The ground slope to the south of the plant is very gentle, approximately 1:400. Figure 7.12 of Reference 46 indicates that for such mild slopes the runup of these waves is less than 10 percent of the incident height, and therefore negligible in effect. Thus the plant is not threatened by wave action at the time of maximum wind velocity.

At the peak of the surge, the average depth of water in the basin is conservatively taken as15 ft. and the significant wave height and period are 4.4 ft. and 3.8 sec. For nearly flat slopes, the height of the breaking wave is approximately 4.0 ft., and the breaking depth is 5.1 ft.[46] Thus the maximum ground elevation on which the significant waves can break is +13.0

ft. MSL. All waves approaching from due south or southwest will break on the railroad embankments or natural ground surfaces. Waves refracted around the relatively high ground on the west side of the site could approach the turbine building from the southeast. Other waves, broken and reformed at lesser heights, could also reach the plant grade.

→(LBDCR 15-026, R309)

Since the Turbine Building is surrounded by flat surfaces, waves will not be induced to break against the walls. At the peak of the surge, the building will be surrounded by water 3.6 ft. in depth. The maximum height of waves, which will not break before reaching the edge of the plant grade, is 0.78 times the depth, or 2.8 ft. Waves of this height or less will reach the walls without breaking, and will be reflected to a maximum of twice their incident height. Using this conservative assumption, the maximum water level is 23.7 ft. MSL (18.1 + [2 x 2.8]).

The pressure exerted against the walls is equal to the static head at the still-water level plus a pressure given in Reference 46 as:

$$P_1 = \frac{1+\chi}{2}\frac{\gamma Hi}{\cosh(2\pi d/L)}$$

where χ is the reflection coefficient, computed assumed as 1.0, Hi is the incident wave height, Υ is the specific weight of water, and L is the significant wavelength to be 66 ft. The value of P_1 is 174 psf. The maximum load is 404 psf (64#/ft.^{3} x 3.6 ft. + 174 psf). The distribution can be conservatively approximated as increasing linearly from zero at 23.7 ft. MSL to 404 psf at ground level, 14.5 ft. MSL.

←(LBDCR 15-026, R309)

2.4.5.4 Resonance

The only resonance phenomenon of possible concern at Waterford 3 is that between a hurricane surge and the natural period of the marshland basin south and east of the site. In the numerical study of the overland hurricane surge, the time between the peak surge at the open coast and the peak at the site was found to be 17.75 hours. If the region of maximum winds traversed the basin in the same time span, full resonance would occur.

The distance from the open coast to the site is approximately 58 miles (50.4 nautical miles). The PMH, at the postulated minimum speed of four knots, travels this distance in 12.6 hours. Thus the surge lags somewhat behind the winds. Partial resonance exists in that a strong forward component of the winds is still driving the surge when it peaks at the site. This is the reason for the unexpectedly high water level of +18.1 ft. MSL. However, a greater degree of resonance could only occur in the event of a hurricane traveling at a speed less than the postulated minimum speed.

2.4.5.5 Protective Structures

The design of safety related structures for protection against flooding and wave action is discussed in Section 3.4. Hydrologic design criteria are summarized in Subsection 2.4.10.

2.4.5.6 PMH Induced Levee Failure

The PMH is capable of producing a stage in the Mississippi River near the site which is one foot higher than the PMF, but only for a brief duration. Due to the likelihood of severe wave activity, however, levee failure adjacent to Waterford 3 must still be considered possible. For the dry bed assumption, the analysis of the latter half of Subsection 2.4.3.7 is repeated, increasing the value of y_0 by one foot. At the limiting value of $y/y_0 = 0.2$, the sum of the depth plus velocity head brings the water level to +25.4 ft. MSL. This increase of 0.8 ft. is still well below the design criterion of static pressure to +30.0 ft. MSL. This case is also presented in Table 2.4-9.

A similar increase in effective water level would be expected in the event of levee failure concurrent with site flooding. However, the surge analysis of Subsection 2.4.3.7 is not repeated because site flooding coincident with the arrival of the PMH surge in the river is not reasonably possible. Hypo Flood 52-A is not of sufficient severity to cause massive levee failure and site inundation. Site flooding from overland surge propagation is possible, but would take place considerably after the peak of the surge in the river has passed. Since only the peak of the river surge produces a stage greater than that considered for the PMF, the analysis is not necessary.

The river surge propagates from Head of Passes to Waterford 3 in four hours. At the end of that time the region of maximum hurricane winds, presuming it travels toward Grand Isle, will not have reached shore. Only when it does, about five hours later, will the peak of the open coast surge begin to travel across the marshes; it will take 17.75 hours to reach the site. The water level at Waterford 3, as computed, first reaches plant grade only some eight hours before the peak arrives. Thus site flooding will not begin until 15 hours after the surge in the river has peaked; site flooding peaks about 23 hours after the river. By this time the river stage will have subsided nearly to normal

The storm would have to accelerate dramatically after passing Head of Passes in order to produce any possibility of the overland surge reaching the site while the river was still above +27 ft. MSL. In this event, the height of the overland surge would drop drastically because of resonance considerations (as discussed in Subsection 2.4.5.4), and Waterford 3 would not be flooded at all.

2.4.6 PROBABLE MAXIMUM TSUNAIII FLOODING

Occurrences of tsunamis are relatively rare in the Gulf of Mexico and Atlantic Ocean as compared to the Pacific Ocean. Between 1800 and 1947 a total of 13 large tsunamis were recorded in the Atlantic Ocean as compared to 148 recorded in the Pacific Ocean (see Reference 47). The oceanic zone of recent earthquake activity nearest to the plant site as shown by Van Dorn in the same reference, is an area in the Caribbean Sea extending eastward from Central America into the Atlantic Ocean through the West Indies and Cuba. This zone is at least 700 miles from New Orleans. The Puerto Rico Trough, which is a part of this zone, is approximately 1500 miles from New Orleans. Any tsunami which might be generated in the Puerto Rico Trough would have to pass through or around Cuba and the West Indies and be reflected off of South America, northward, to reach the site. The resulting wave diffraction and reflection would greatly attenuate the tsunami before it reached the Gulf Coast of the United States.

The only recorded earthquake in the Gulf of Mexico within 200 miles of the site occurred on November 5, 1963 (Refer to Subsection 2.5.2.1). The epicenter of this earthquake was 191 miles (27.8^{o} north latitude and 92.4^{o} west longitude) from the site and had a magnitude of 4.8. According to studies performed by lida, as presented in Reference 48, an earthquake would have to have a magnitude of at least 6 to generate a tsunami.

It is concluded, therefore, that the Gulf Coast near the site will not experience any significant tsunami flooding. Any tsunami flooding effects that may be postulated will be minor in comparison to the hurricane surge flooding described in Subsection 2.4.5.

2.4.7 ICE EFFECTS

→(LBDCR 15-026, R309)

The appearance of ice on the lower reach of the Mississippi River is a rare occurrence, especially below the vicinity of Baton Rouge (Mile 228.4 AHP). The mild to moderate quantity of drift ice which has been observed in this region has an estimated frequency of occurrence of two or three times in the past 100 years, and has never resulted in ice jams which might have caused some visible damage or impaired river navigation.[49,50,51,52] This drift ice originates from the breakup of the massive ice flows which occur in the upper reaches of the Mississippi River. These ice flows have drastically hampered navigation in the upper reaches, and have caused U. S. Coast Guard to periodically close the river to such activities. These ice flows, however, lack a history of having caused any extensive or severe damage to waterfront property in the lower Mississippi.

Documentation of these hydrologic events is lacking, but information has been obtained from experienced hydrologists working for different States and Federal agencies. They recall past occurrences to have taken place during the winters of 1890, 1940, and 1976 through 1977. With respect to the latest date from mid-January to mid-February, the Coast Guard had officially closed the Mississippi River to navigation from Cairo to St. Louis because of ice flow blockage.

Considering all available information, it is concluded that the Waterford site will not experience any difficulties or problems which might arise from ice flooding or ice flow blockage.

2.4.8 COOLING WATER CANALS AND RESERVOIRS

There are no cooling water canals or reservoirs at the plant site.

2.4.9 CHANNEL DIVERSIONS

Historically, Old River has presented the greatest potential for diversion of the Mississippi River. Old River is a seven-mile stream that connected the Mississippi with the Red and Atchafalaya Rivers and was formed in 1831 when one of the loops of the Mississippi was cut off to shorten navigation. The Atchafalaya began enlarging itself through the diversion of greater amounts of Mississippi flow and if left alone, would have become the main channel of the Mississippi River. To prevent this from happening, and to aid in flood control, the Corps of Engineers built the Old River control structure which was completed in 1963.

←(LBDCR 15-026, R309)

During the flood of 1973, the south approach wing wall of the structure collapsed due to scouring of the riprap protection and removal of the alluvial foundation material. Quick remedial action by the U. S. Army Corps of Engineers prevented the structure from being flanked by the river flow and collapsing. If the structure had failed, the Atchafalaya River could have become the main channel of the Mississippi River.

Measures to prevent a future similar occurrence have been proposed by the Corps of Engineers and even a replacement structure has not been ruled out.[53] Should a diversion ever occur and the river's supply of fresh water be stopped, the bottom of the pump intake bell is still -13 ft. MSL and the Circulating Water System could still draw water. As explained in Subsection 2.4.11 the Component Cooling Water System is the ultimate heat sink for the plant, and the Circulating Water System is not necessary for dissipating heat during an emergency shutdown condition.

2.4.10 FLOODING PROTECTION REQUIREMENTS

All safety related equipment is housed within the Nuclear Plant Island Structure (NPIS). The NPIS is a reinforced concrete box structure with solid exterior walls and is flood protected up to El. +30.0 feet MSL. (See note under Subsection 2.4.1.1.)

→(EC-42115, R307, LBDCR 15-026, R309)

The site slopes downhill from the Nuclear Plant Island Structure to the west, south and east, and is at a plant grade of 17.5 ft. MSL between the NPIS and the Mississippi River levee. Because of this topography, only two events can produce flooding of the site: a slow-moving PMH approaching from the south, and failure of the Mississippi River levee adjacent to the plant. The peak stillwater level from the overland PMH surge is 18.1 ft. MSL. Wind waves from the south would have a maximum height of 2.8 ft. Upon reflecting from the NPIS walls, such waves produce a maximum water level of 23.7 ft. MSL (Subsection 2.4.5.3). The maximum combined static and dynamic load would increase linearly from zero at El. 23.7 to 404 psf at El. 14.5. For a Mississippi River flood greater than the Project Design Flood, producing a river stage of 27.0 ft MSL, instantaneous levee failure would result in a maximum water level on the north wall of the NPIS of 24.6 ft. MSL, including the dynamic effect of velocity head (Subsection 2.4.3.7). The maximum combined static and dynamic load would increase linearly from zero at El. 24.6 to 443 psf at El.17.5. In the event of a PMH coincident with Hypo Flood 52A, the maximum river stage would be 28.0 ft MSL and the maximum water level at the NPIS resulting from instantaneous levee failure would be 25.4 ft. MSL (Subsection 2.4.5.6). The maximum combined static and dynamic load would increase linearly from zero at El. 25.4 to 493 psf at El. 17.5. If a river stage of 30.0 ft. MSL at the top of the levee is considered to be possible, the maximum water level at the NPIS resulting from instantaneous levee failure would be 27.6 ft. MSL. The maximum combined static and dynamic load would increase linearly from zero at El. 27.6 to 630 psf at El. 17.5. All of these conditions result in water levels, including dynamic effects, below the design criterion of flood protection to El. 30 ft. MSL. The effects of "splash" are negligible in all cases. Technical Requirements Manual 3.7.5 describes procedures for monitoring flood danger and securing all openings below El. 30.0 ft. MSL when site flooding becomes a possibility. The design flood level of 30.0 ft. MSL was used in calculations of load combinations.

←(EC-42115, R307, LBDCR 15-026, R309)

→(EC-45161, R308)
The highest depth of water calculated inside the NPIS resulted from the Probable Maximum Precipitation on the Cooling Tower Areas. Inside the open cooling tower areas, all safety-related equipment is located or protected such that no damage will result from flooding. The minimum elevation of safety-related equipment is approximately 1.657 ft. above the mat. The critical height of equipment in the DCT cubicles (DCT sump pump motors) is approximately 1.417 ft above the mat.
Refer to Subsection 2.4.2.3.
←(EC-45161, R308)

Roof design has been reviewed according to the criteria for load combinations listed in Table 3.8-39, Formula 5. Both the six hour PMP giving the maximum intensity, and the 48 hour PMP giving the maximum accumulation have been considered. All roofs of safety related structures can safely store the maximum possible ponding resulting from the PMP. Figure 2.4-8 shows the locations and sizes of all roof drains and scuppers, and the heights of parapets.

2.4.11 LOW WATER CONSIDERATION

2.4.11.1 Low Flow in Streams

The intake structure for Waterford 3 will not be required to operate under probable minimum low flow conditions since it is not safety-related.

A frequency analysis from a hydrologic investigation which was conducted by the Louisiana Department of Public Works in consortium with the USGS[55] reports that the 95 percent exceedence flow (that flow which will be equalled or exceeded 95 percent of the time) is 131,000 cfs.

Recently, the 95 percent exceedence flow for the lower Mississippi River has been updated to 140,000 cfs for the USGSM, for a hypothetical gauging station located midway between Red River Landing and Tarbert Landing (see Table 2.4-12).

The low flow date from Red River Landing reflects the earlier period 1930 through 1963, prior to the completion of the Old River diversion channel. This data was combined with the next 12 years of data from Tarbert Landing approximately five miles further upstream. The increase in the 95 percent exceedence flow may be explained by the completion of the Old River control structure and the additional construction of storage reservoirs on tributaries. These controls are designed to sustain a minimum flow of 100,000 cfs during low flow periods. i.e. the probability of a low flow below 100,000 cfs is practically zero (Figure 2.4-26). This flow is assumed to be at least as severe as the 100 yr drought.

A water surface profile, using cross section data from the Corps of Engineers for a flow of 100,000 cfs, was computed from the mouth of the Mississippi River to the Waterford 3 Site. This resulted in a calculated low water level of El. -0.43 ft. MSL at the intake structure.

2.4.11.2 Low Water Resulting from Surges, Seiches or Tsunami

As previously indicated, the Waterford 3 intake structure is not a safety-related structure and consequently will not need to operate under an induced low-flow condition.

2.4.11.3 Historical Low Water

The minimum daily flow of 75,000 cfs (stage 1.15 ft.) was calculated by the Army Corps of Engineers as having occurred on November 4, 1939, at Red River Landing (mile 302.4 AHP), a gauging station located on the Mississippi River below the Old River diversion channel near Coochie, Louisiana.[55] During this period, 13,400 cfs were being diverted by the Old River diversion channel into the Atchafalaya River, and the minimum daily flows at Vicksburg and Natchez, Miss., were 102,000 and 100,000 cfs, respectively.

2.4.11.4 Future Controls

The Louisiana Department of Public Works has conducted an investigation[6] to determine the state's projected surface water requirements to the year 2020.

Their study concludes that the Southeast Sector of Louisiana has and will continue to have the maximum surface water requirements for all years.

Of the state's total surface water requirements, this sector consumed 45 percent in 1970 and the projected estimate for 2020 is 43 percent, with industrial and thermoelectric categories requiring the largest portion.

For communities below Baton Rouge, the Mississippi River is the principal source of all municipal and industrial surface water requirements. Particularly for the southernmost areas, such as Plaquemines Parish, the quality of this water will, during periods of low flow, suffer from salt water intrusion, but the overall picture indicates that this sector should not experience any surface water supply problems. There are, at this time, no future plans for the implementation of additional discharge controls on the lower Mississippi River.

2.4.11.5 Plant Requirements

→(EC-530, R303, LBDCR 15-026, R309)

There are normally no safety related water requirements from the Mississippi River for Waterford 3. The Component Cooling Water System utilizes dry-wet cooling tower combinations as the ultimate heat sink, as described in Subsection 9.2.5. Mississippi River water may be utilized as a source of makeup water to the wet cooling tower basins following a tornado event as discussed in Subsection 9.2.5.3.2 and 9.2.5.3.3. The Circulating Water System is discussed in Subsection 10.4.5.

2.4.11.6 Heat Sink Dependability Requirements

The design of Waterford 3 is consistent with applicable recommendations of Regulatory Guide 1.27, Rev. 2, Ultimate Heat Sink for Nuclear Power Plants (1/76). The UHS utilizes replenishment from an alternative water supply (on-site water sources and/or Mississippi River water) to ensure cooling capacity for 30 days and beyond in response to a design basis tornado event. Subsection 9.2.5 provides further discussion of the Ultimate Heat Sink.

←**(EC-530, R303)**

2.4.12 DISPERSION, DILUTION, AND TRAVEL TIMES OF ACCIDENTAL RELEASES OF LIQUID EFFLUENTS IN SURFACE WATERS

The possibility of an accidental release of radioactive effluents by Waterford 3 reaching a surface water body is virtually nonexistent. The waste storage tanks are within the Reactor Auxiliary Building which is a seismic Category I structure. Should a tank rupture occur, the released effluent would drain to floor sumps inside the Reactor Auxiliary Building, and

←(LBDCR 15-026, R309)

from there be pumped back through the Liquid Waste Management System.
→(LBDCR 15-026, R309)
At the lower elevation, a fissure will allow water to seep in rather than out.
←(LBDCR 15-026, R309)

If a fissure should exist in the walls or floor of the building high enough to allow the escape of the released effluent, the effluent would disperse throughout the sand back filled area around the foundation. The surficial soil surrounding the backfilled area consists of recent alluvial deposits of soft and silty clays of low permeability. Well point yields during the dewatering operation showed that there was no hydraulic connection between the excavated area and the Mississippi River (refer to Subsection 2.4.13.3). Hence, radioactive effluents which escape through the walls would not find a path to the river.

The only conceivable way that untreated radioactive effluent could be released to the Mississippi River would be through the Liquid Waste Management System. Liquid waste samples are analyzed prior to any release to determine if releases are within acceptable limits. All bypass lines around the purification equipment have normally closed valves, and all lines which bypass the waste concentrators also have closed valves. All waste must be pumped to the Circulating Water System discharge; thus, positive operator action is required to line up tanks and start pumps. The Liquid Waste Management System is further described in Section 11.2.

Studies of dye dispersion and travel time in the Mississippi River[56,57] show that in the event of an accidental release, the concentration of any radioactive effluents released at the Waterford 3 site will be greatly dimished by the time the effluent reached the intake of any water users. The relative concentrations vs. distance downstream for the release of a unit of a conservative effluent from Waterford 3, for river discharges of 200,000 cfs and 400,000 cfs, is shown in Figure 2.4-27. Downstream water users and their locations relative to the site are presented in Table 2.4-1.

2.4.13 GROUNDWATER

This subsection presents conditions, sources, and usage of aquifers in the region, the site area, and at the site. The general areas which correspond to these divisions are depicted in Figure 2.4-28.

Groundwater in southeastern Louisiana is available in deltaic and shallow marine deposits of Tertiary and Quaternary age. The major aquifers in this region are unconsolidated sands that dip southward. In general, these sand deposits are separated and confined by relatively impermeable clays and silts.

Major water-bearing zones can be correlated in a northwest-southeast direction along the Mississippi River between Baton Rouge and New Orleans.

The connate water within the aquifers in southeastern Louisiana is generally brackish or salty. Fresh water is found only near areas of recharge where it has displaced the salty connate water.

Because of the southerly dip of the aquifers in the region, deep aquifers approach the land surface further to the north than shallow aquifers. Since the topography of the region rises from south to north, the recharge areas for the deeper aquifers are at a higher elevation than those for the shallower aquifers. This circumstance induces, under natural conditions, a general piezometric gradient which falls from north to south and also causes an increase in piezometric head with depth at a given location.

2.4.13.1 Description and Onsite Use

2.4.13.1.1 Regional Conditions

The region surrounding the site is depicted in Figure 2.4-28. The principal aquifer system which exists within the site are in order of increasing depth:

a) The Shallow Aquifers
b) The Gramercy Aquifer
c) The Norco Aquifer
d) The Gonzales-New Orleans Aquifer

The aquifer systems are named for areas in which they are the principal aquifer. The shallow aquifers include point bar deposits and other shallow deposits of sand. The Gramercy aquifer is the principal freshwater bearing sand in the Gramercy area and has previously been called the “200-ft.” sand. The Norco aquifer is the principal aquifer in the Norco area and has been called the “400-ft.” sand in New Orleans. The Gonzales-New Orleans aquifer is a thick sand which underlies the Norco aquifer in the region, and has previously been called the Gonzales aquifer or the “700-ft.” sand.

a) The Shallow Aquifers

Shallow isolated sands, isolated point bar deposits, and abandoned distributary deposits are collectively described as shallow aquifers. Localized sand deposits below depths of about 150 ft. have small yields of poor quality water and are not recognized as important aquifers in the region.[62,65] The shallow deposits occur frequently in the Mississippi River deltaic plain but are not interconnected regionally.

→(LBDCR 15-026, R309)

Small quantities of poor quality water are available in point bar deposits that line the inside bends of the Mississippi River[65]. The water in these deposits is low in chloride but is characteristically hard and usually has a high iron content.[62] Point bar deposits consist chiefly of well-sorted fine sands and silts with a maximum thickness of about 130 ft.[62,63] The point bars are typically overlain by about 20 to 30 ft. of natural levee material.

Estimates of permeability, based on texture of the material comprising the deposits, are generally low, and consequently, the sustained yield of wells in point bar deposits is low. The highest reported yield of a well in a point bar deposit is 50 gpm; however, this well was located in a point bar deposit of unusually coarse texture. More typical wells in point bar deposits have yields of only a few gpm.

←(LBDCR 15-026, R309)

Small supplies of potable water are sometimes found in abandoned distributary channel deposits. These deposits consist chiefly of silty, sandy soils, and are generally more permeable than other surface materials. Rainwater directly recharges the distributary channel deposits and may locally flush or displace brackish or salty water from other shallow aquifers which are connected to the distributaries.[65] Large quantities of fresh water cannot be developed in these deposits because salt water which underlies or is adjacent to would move into the area after a period of continuous pumping.

b) The Gramercy Aquifer

The Gramercy aquifer is the principal fresh water aquifer in the Gramercy area but has little use in the region. The aquifer is a medium to very fine grained sand in the New Orleans area.[61] Its grain size increases to the west toward Norco[62] and decreases to the south toward the Gulf.

The Gramercy aquifer is continuous in the Gramercy area, but is discontinuous in both the New Orleans and Norco areas. The aquifer generally increases in thickness in north to south direction around both Norco and New Orleans.[62, 65] The aquifer is about 100 ft. thick in the Norco area[62] and ranges from 30 to 150 ft. in thickness in the New Orleans 61 Values of transmissivity of the Grammercy aquifer range from 20,000 gpd/ft. in the vicinity of New Orleans to as high as 240,000 gpd/ft. near Norco.[62, 65]

Fresh water (less than 250 ppm chloride) occurs in the Gramercy aquifer in the Gramercy area, and in the northwest corner of Jefferson Parish; fresh water may also exist in the aquifer near Lake Cataouatche and in isolated areas along the Mississippi River.[62] Little use has been made of the Gramercy aquifer as a water supply in the New Orleans and Norco areas[62, 65] because the water of both areas is generally high in magnesium and calcium. The salinity of the water increases in a southerly direction.

→(LBDCR 15-026, R309)

c) The Norco Aquifer

In the New Orleans area, the Norco aquifer is a medium to fine grained sand; in the Norco area, it grades to a medium to coarse sand and is the principal aquifer.[62] In eastern New Orleans, the Norco aquifer pinches out; to the south of the New Orleans area, the aquifer thickens and grades into thin sand stringers. The Norco[61] aquifer thins and becomes unidentifiable to the north beneath Lake Pontchartrain.[62] Thicknesses of the Norco aquifer vary widely, ranging from 25 to 300 ft. in the Norco area, and from 95 to 172 ft. in the New Orleans area where it averages approximately 120 ft.[65] The top of the Norco aquifer in both the New Orleans and Norco areas is encountered between depths of about 30D to 400 ft.[62, 65]

The Norco aquifer dips about 10 ft. per mile to the south in the Norco area; in the New Orleans area, the regional dip Gulfward may decrease about five ft. per mile.[61, 62] The Norco aquifer is usually separated from the overlying Gramercy aquifer by clay beds with interbedded sand stringers in the New Orleans area.[61] In the Norco area, a large area of convergence exists between these aquifers.

←(LBDCR 15-026, R309)

Data from pumping tests in the Norco aquifer indicate that the transmissivity increases from 50,000 gpd/ft. in the New Orleans area to as much as 225,000 gpd/fft. in the Norco area, where the aquifer is continuous.[62, 65] Well yields as high as 3,000 gprn have been obtained from wells tapping the Norco aquifer in the vicinity of Norco; however, the yield of most wells in the area range from 1,000 to 1,500 gpm.

Limited use is made of the Norco aquifer in the New Orleans area; concentrations of chloride are generally greater than 250 ppm except in the extreme northwest portion of Jefferson Parish where fresh water occurs.[65]

→(LBDCR 15-026, R309)

Heavy pumpage in the Norco area and hydraulic connections between the Gramercy and Norco aquifers have resulted in a mixing of the water in these aquifers. Salty water from the Gramercy aquifer has moved into the Norco aquifer. Hard water in point bar deposits, in turn, has replaced the salty water in the Gramercy aquifer.[62]

←(LBDCR 15-026, R309)

d) The Gonzales-New Orleans Aquifer

In the New Orleans and Norco areas, the Gonzales-New Orleans aquifer is a fine grained quartz sand of uniform texture. [61, 62] It is the principal aquifer in the New Orleans area[61] and in the Geismar-Gonzales area.[64] The Gonzales-New Orleans aquifer is present over a large region along the Mississippi River from the Baton Rouge area to the New Orleans area, and northeast of New Orleans to the Fort Pike area.[59] Correlation of the aquifer in a north-south direction is difficult since thicknesses vary greatly in this direction. The Gonzales-New Orleans aquifer correlates with a zone of shallow aquifers north of New Orleans beneath Lake Pontchartrain; however, this zone contains coarser materials.[58] In the New Orleans area, the thickness of the Gonzales-New Orleans aquifer generally ranges between 100 and 200 ft. and averages 175 ft.[65] The average thickness of the Gonzales-New Orleans aquifer increases to about 225 ft. in the Norco area where thicknesses range from 175 to 325 ft.[62] The depth to the top of the aquifer in the New Orleans Norco area ranges from about 450 to 800 ft. In most of the Norco area, the Gonzales-New Orleans aquifer is separated from the overlying Norco aquifer by 200 to 300 ft. of clay with interbeds of sand.[62] The Gonzales-New Orleans aquifer dips to the south at rates ranging from 20 ft. to as much as 50 ft. per mile.[62, 65]

Values of transmissivity, based on five pumping tests in the New Orleans area, range from 90,000 gpd/ft. to 180,000 gpd/ft.[65] Two values available in the Norco area fall within this range (146,000 and 150,000). Higher values of transmissivity are noted in the Geismar-Gonzales area, where the aquifer ranges in texture from a fine to very coarse sand and gravel. [64]

Fresh water (less than 250 ppm chloride) in the Gonzales-New Orleans aquifer is generally encountered north of the Mississippi River in the region. The fresh water in the New Orleans area is not entirely satisfactory for public supply because the water has a yellow color of organic origin. [65] In the Norco area, the fresh water of the aquifer is soft and of the mixed sodium bicarbonate chloride type, and has little color.[62]

2.4.13.1.2 Site Area Conditions

The site area is outlined in Figure 2.4-28. The site area coincides with the “Norco area”, the groundwater resources of which are described by Hosman.[62] The shallow aquifers, the Gramercy aquifer, the Norco aquifer, and the Gonzales-New Orleans aquifer are present in the site area; the Norco aquifer, is the principal aquifer in the area. The area is hydrologically complex because of hydraulic interactions between the major aquifers.

→(LBDCR 15-026, R309)

a) The Shallow Aquifers

The near surface, bodies of sand which form shallow aquifers in the site area are limited and irregular in areal extent and occur at depths generally less than 150 ft. The only shallow sands which are extensive enough to provide small quantities of water are point bar deposits and abandoned channel deposits of the Mississippi River.

←(LBDCR 15-026, R309)

The point bar deposits accumulate on the inside of river bends in the site area as shown in Figure 2.4-29. The point bar deposits in this area are overlain by 20 to 30 ft. of natural levee deposits with a maximum thickness of about 130 ft.[62] Water levels in the point bars follow the stage of the Mississippi River, being higher than the river at low stage and slightly lower at high stage. The point bars in the site area are not important aquifers because the water is very hard and high in iron content.

The permeability of shallow aquifers in the site area is estimated to be low (about 100 gallons per day per sq. ft.) based on the texture of the deposits.[62] The low permeability, poor quality of water, and limited extent of the shallow aquifer restricts their utility in the site area. The shallow aquifers are significant only as connections between the Mississippi River and deeper aquifers.

b) The Gramercy Aquifer

In the site area, the Gramercy aquifer is irregular in thickness and discontinuous. The extent and configuration of the top of the aquifer are shown in Figure 2.4-30. The aquifer dips and thickens to the south of the site area. The top of the aquifer occurs at about -200 ft. MSL beneath the southern portion of the site property and is about 100 ft. thick. The aquifer is split, thin, or absent immediately north of the site property.

In the vicinity of Hahnville, the Gramercy aquifer is in contact with a point-bar deposit. This connection permits flushing of the Gramercy aquifer which contains salty water in much of the surrounding area. Fresh water (chloride content 250 ppm or less) is also encountered in the Gramercy aquifer in the southwest portion of the site area as shown in Figure 2.4-30. The quality of water in the Gramercy aquifer varies more than in the Norco and the Gonzales-New Orleans aquifers in the site area.

Well yields from the Gramercy aquifer in the site area range from several hundred to more than 1.000 gpm. A transmissivity on the order of 150,000 gpd per ft. is indicated for the aquifer in the vicinity of Destrehan.[62]

c) The Norco Aquifer

The Norco aquifer is continuous throughout the site area and varies from 25 to 300 ft. in thickness. The Norco and Gramercy aquifers are probably in contact in areas shown in Figure 2.4-31. The Norco aquifer is separated in most of the site area from the underlying Gonzales-New Orleans aquifer by 200 to 300 ft. of clay. Discontinuous sand beds are found within this clay layer. The top of the aquifer occurs at about -325 ft. MSL beneath the site and is about 125 ft. thick. The regional thickening and dip of the aquifer is to the south.

The interaction with the overlying Gramercy aquifer has a profound effect on the Norco aquifer both under natural and man-made conditions. The large southwest loop in the fresh-salt water interface shown in Figure 2.4-31 is most likely related to the presence of the large area of convergence of the aquifers southwest of the site. Prior to pumping in the Norco aquifer, water moved upward from the aquifer through the overlying confining beds producing a natural flushing action as fresh water moved downdip. A greater volume of fresh water was discharged vertically in the large area of aquifer convergence than in areas where the confining beds are present, and at a much higher rate of movement.

The hydrostatic pressures in the Gramercy and Norco aquifers have been reversed by large scale pumping activities which began at Norco in 1920. Water levels are now higher in the Gramercy aquifer than those in the Norco aquifer. Therefore, groundwater presently moves from the Gramercy aquifer, through the area of convergence, and into the Norco aquifer. The body of fresh water in the aquifer east of Luling, shown in Figure 2.4-31, is probably related to upward movement of water from the Gonzales-New Orleans aquifer through a leaky zone in the overlying confining bed.

The transmissivity of the Norco aquifer in the site area is about 200,000 to 225,000 gpd/ft. and the permeability is about 1,600 to 1,800 gpd/ft^2. Most wells in the Norco aquifer yield from 1,000 to 1,500 gpm and most specific capacities range from 45 to 75 gpm/ft. [62]

d) The Gonzales-New Orleans Aquifer

The Gonzales-New Orleans aquifer is continuous in the site area and ranges from less than 175 to more than 325 ft. in thickness. The occurrence of sands in the overlying clays significantly alter the effective thickness of the aquifer. The top of the aquifer occurs at about -600 ft. MSL beneath the site and is about 250 ft. thick. The configuration of the top of the aquifer is shown in Figure 2.4-32.

The distribution of fresh and salty water in the aquifer appears to result from natural flushing from north to south (Figure 2.4-32). No direct hydraulic connection with other aquifers is known in the area and differences in the chemical quality of the water are gradational.

In the area of the site, the transmissivity of the Gonzales-New Orleans aquifer is lower than that of the Norco aquifer, averaging about 148,000 gpd/ft. The

permeability is on the order of 680 gpd/ft.^{2} and most yields of wells are between 1,000 and 1,500 gpm. The lower permeability and transmissivity of the aquifer compared to that of the Norco aquifer is attributed to finer grained sands which compose the Gonzales-New Orleans aquifer.

→(LBDCR 15-026, R309)

2.4.13.1.3 Groundwater Conditions at the Site

The Waterford site is located on the west bank of the Mississippi River about three miles west of Taft and encompasses over 3000 acres with approximately 7500 ft. of river frontage. Topographically the area is relatively flat with an elevation of +12 ft. MSL. The land slopes slightly downward away from the river levee. The property to the rear of the plant location itself, once a swamp area, has been reclaimed.

A generalized cross section through the site is presented in Figure 2.4-33. The site is immediately underlain by deposits of clay, silt and sand of recent geological age (Zone 1). Based on information obtained from piezometric levels measured since June 1972, this formation is discontinuous and unresponsive to fluctuations in the level of the Mississippi River.

Fifty feet beneath the Recent deposits is an aquiclude of fairly uniform Pleistocene clay (Zone 2) with occasional discontinuous sand lenses. The reactor foundation mat bears upon the Pleistocene clay at elevation -47 ft. MSL. As evidenced in the detailed subsurface investigations this zone exhibits an average permeability of about 10^{-8} cm/sec.

A continuous dense to medium dense silty sand layer (Zone 3) with some clay and approximately 19 ft. in thickness is situated immediately beneath the uppermost Pleistocene clay, starting at elevation -89 ft. MSL. This layer exhibits an average permeability of about $3.0x10^{5}$ cm/sec. in laboratory tests. It is important to note that the Mississippi River immediately north of the plant area has thalweg depth of 122 ft. adjacent to the site. Thus, the groundwater regime of the upper three strata described above, on the south bank of the river will not affect, nor be affected by, the shallow aquifers and groundwater regime on the northern bank.

A stiff clay stratum (Zone 4), from elevation -108 ft. MSL to elevation -330 ft. MSL, behaves as a local aquiclude. The layer is soft at the upper contact with Zone 3 and has a continuous sand layer approximately 10 ft. in thickness and located at approximate elevation -240 ft. MSL. Detailed subsurface investigations indicate that this layer correlates locally with the Gramercy aquifer in a lateral facies change to the southwest of the Reactor Building. The Gramercy aquifer is encountered at elevation -190 ft. MSL to the south-southwest of the Reactor Building in Corps of Engineers test hole SC-93.

Detailed subsurface investigations indicate that the Norco aquifer is locally manifested as a dense silty sand beneath an approximate elevation of -330 ft. MSL.

The Norco aquifer is the only regional aquifer encountered in the sub-surface investigation beneath the site. The Norco aquifer is separated from all the sand deposits encountered beneath the site by the stiff clays encountered in Zone 4. There is evidence in observation well SC-82 (since abandoned), nearby the site excavation, that water levels within the Norco aquifer were influenced by pumping the Norco Industrial Area.

←(LBDCR 15-026, R309)

2.4.13.1.4 Onsite Use of Groundwater

Neither the existing fossil nor proposed nuclear generating facilities utilize groundwater. All safety related water requirements are met by intake from the Mississippi River. Water not directly taken from the Mississippi River is supplied from St. Charles Parish mains.

2.4.13.2 Sources

2.4.13.2.1 Region and Site Area

This subsection examines sources and use of the shallow aquifers, the Gramercy aquifer, the Norco aquifer, and the Gonzales-New Orleans aquifer in the region and site area.

→(LBDCR 15-026, R309)

Recharge to the various aquifers may be derived by several mechanisms: (a) from direct infiltration of surface waters and precipitation, (b) from movement of water through areas of convergence between the aquifers, (c) from vertical leakage through confining beds in response to differences in hydrostatic heads that exist between the aquifers, and (d) from downdip movement of water in aquifers that crop out, or are connected to sands that crop out, north of Lake Pontchartrain.

Prior to the inception of heavy pumping in the New Orleans and Norco areas, groundwater movement in the regional aquifers was generally downdip to the south. As groundwater usage has increased, the direction of movement has been altered and is now generally toward the major centers of pumpage. An increase in vertical leakage through the confining beds has also occurred in some areas where head differentials between adjacent aquifers have resulted from heavy pumpage from one aquifer.

a) The Shallow Aquifers

Water levels in shallow aquifers downstream of the Baton Rouge area closely follow the stage of the Mississippi River. Water from the Mississippi River seeps into shallow aquifers during periods of high river stage and from these aquifers into the river during periods of low river stage.

→(DRN 01-464, R11-A)

During periods of low river stage, a transient groundwater divide may be created in shallow aquifers. Recharge to the shallow aquifers may also be provided by vertical leakage from underlying aquifers of higher hydrostatic head, and by direct infiltration of rainwater.

←(DRN 01-464, R11-A, LBDCR 15-026, R309)

The shallow aquifers in the site area are used for little other than a source of small supply for stock wells because of poor quality water. The potential for development of these aquifers is slight; their utility is restricted by their limited extent, poor quality water, and low permeability.

b) The Gramercy Aquifer

Recharge to the Gramercy aquifer is derived from the river via hydraulic connections to the shallow sands overlying this aquifer. Recharge is also obtained from vertical leakage from overlying and underlying aquifers.

In both the New Orleans and Norco areas, piezometric levels in the Gramercy aquifer near the Mississippi River closely follow the stage of the river. As the distance from the river increases, the range of the water level fluctuation decreases, unless influenced by external factors such as pumping.

Pumping from the Gramercy aquifer has created localized cones of depression in the piezometric levels in the New Orleans area; however, these depressions do not extend far from the centers of pumping. A more extensive depression of piezometric levels of the Gramercy aquifer has occurred in the site area (Figure 2.4-34). This depression is not attributed to pumpage from the Gramercy aquifer[62, 65], but rather to vertical movement of water from the Gramercy aquifer into the Norco aquifer in an area of convergency of these two aquifers. This vertical leakage from Gramercy aquifer has been induced by heavy pumpage from the Norco aquifer with a consequent lowering of the piezometric surfaces of the Gramercy aquifer.

Total withdrawals from the Gramercy aquifer in the site area are very small, probably less than 0.1 mgd.[62] The few wells that utilize the aquifer in the area are small-capacity wells for domestic supplies and stock watering. The quality of water in the aquifer is a limiting factor in development.

c) The Norco Aquifer

The natural regional movement of water in the Norco aquifer toward the south has been altered by the heavy pumping at Norco. Most of the water in the aquifer in the site area is moving toward the Norco pumping center.

The Norco aquifer is hydraulically connected to, and recharged by, the Gramercy aquifer in the site area; however, the two aquifers generally are separated by clay beds interbedded with sand in the New Orleans area. Recharge to the Norco aquifer may be augmented in an area near Luling by indirect hydraulic connections to the underlying Gonzales-New Orleans aquifer.[62] This connection is indicated by a section of fresh water in the Norco aquifer that corresponds to fresh water in the underlying Gonzales New Orleans aquifer[62], and could result from low confining ability of the aquiclude. which is known to be sand in this area.

Water levels in the Norco aquifer reflect heavy industrial pumpage around Norco, as seen in Figure 2.4-35. Water levels quickly adjust to changes in pumpage because of the nearby source of recharge afforded by the large area of convergence with the overlying Gramercy aquifer. Water levels in the Norco aquifer, based on 1960 data, are as low as -50 ft. MSL in the vicinity of Norco and -15 ft. MSL in the New Orleans area. Figure 2.4-35 shows the approximate 1960 piezometric levels in the aquifer.

Most of the groundwater used in the Norco area is obtained from the Norco aquifer, the principal aquifer in the area. Water usage in the Norco area has fluctuated over the past thirty years. Withdrawals were about 7.5 mgd in 1942 and increased to 17.5 mgd in 1950 and to about 11 mgd in 1976. Usage decreased to about 9.5 mgd in 1965, and has generally remained below 11 mgd through 1976[60,62] The Norco aquifer is used almost exclusively for industrial purposes; the only public use is in the vicinity of

Laplace, Louisiana, where about 0,066 mgd is withdrawn.[62]

The development potential of the Norco aquifer is large and is expected to be limited primarily by water quality considerations. Because of the high transmissivity of the aquifer and the nearby sources of recharge, water levels in the aquifer do not fluctuate a great deal in response to increases and decreases in pumping. Water levels in the Norco aquifer during a period from about 1943 to 1976 are shown in Figure 2.4-36. As shown in Figure 2.4-36, groundwater levels in the Norco aquifer have gradually risen since 1965 as a result of the decreased usage of groundwater at the Norco refinery.

d) The Gonzales-New Orleans Aquifer

The Gonzales-New Orleans aquifer in the Norco and New Orleans area is primarily recharged to the north of Lake Pontchartrain. Beneath Lake Pontchartrain, the Gonzales-New Orleans aquifer correlates with a zone of shallow aquifers which crop out to the north of the lake.[58] These shallow aquifers receive recharge from precipitation and surface waters in a large area general between Covington, Louisiana and the state of Mississippi.[61] Water in these shallow sands moves downdip into the Gonzales-New Orleans aquifer, and thence south into the New Orleans and Norco areas.

Recharge to the Gonzales-New Orleans aquifer is also derived by downward leakage of water directly into the aquifer through shallow clays that exist south of the Covington area and beneath Lake Pontchartrain. [58] Vertical leakage into the Gonzales-New Orleans aquifer also occurs in local areas from underlying aquifers through aquicludes which have low confining ability due to high sand contents.

→(LBDCR 15-026, R309)

Original water movement in the Gonzales-New Orleans aquifer in the New Orleans area was generally from north to south. Groundwater in the aquifer now moves toward the heavy industrial pumping center at New Orleans where the potentiometric surface in early 1974 was as low as -120 ft. MSL.[67] The configuration of the potentiometric surface in the Gonzales-New Orleans aquifer in the site area in the fall of 1965 is shown in Figure 2.4-37. Pumpage in the site area has created local cones of depression in the potentiometric levels of the aquifer at Norco and at Kenner. The local cones of depression in the site area are superimposed on the edge of the large regional depression in the potentiometric surface caused by withdrawals at New Orleans.

Water levels in the Gonzales-New Orleans aquifer show a definite response to changes in the pumping rate in New Orleans. During the period from 1906 to 1963, water levels in well OR-42, near the downtown center of pumping in New Orleans, declined at an average rate of about 1.5 ft. per year.[65]

A hydrograph showing water level in the Gonzales-New Orleans aquifer from 1961 to 1976 is shown in Figure 2.4-38. This hydrograph indicates that water levels are not continuing to decline at the rate observed from 1906 to 1963, but leveled off during the period from 1968 to early 1974, and have begun to rise from late 1974 to the present as a result of decreased pumpage.

←(LBDCR 15-026, R309)

Quantities of water pumped from the Gonzales-New Orleans aquifer in the Geismar-Gonzales area are relatively small and water levels recover quickly to static levels because of the high transmissivity of the aquifer in that region.[64] The water levels in the Gonzales-New Orleans aquifer in the Norco and New Orleans areas do not stabilize as readily as do those in the Geismar-Gonzales area, probably because of lower values of transmissivity and the absence of a significant local source of recharge.

→(DRN 02-123)

Pumpage from the Gonzales-New Orleans aquifer in the site area is about six mgd.[62] The water is pumped from the aquifer for irrigation and industrial purposes. The potential of the aquifer in the area is governed primarily by water quality tolerances, as heavy pumping near the fresh-salt water interface will be accompanied by increased salinity. Large scale developments in the aquifer are not likely in the site area because of the advantages of a higher transmissivity in the overlying Norco aquifer. Also, the effects of pumping from the Gonzales-New Orleans aquifer are more extensive than are effects of pumping from the Norco aquifer.

←(DRN 02-123)

2.4.13.2.2 Existing Groundwater Users and Related Effects

→(DRN 01-464)

Groundwater usage inventories were conducted in the region surrounding the Waterford site. A literature reconnaissance utilizing information available from Federal and State sources was completed for St. Charles Parish and portions of Jefferson Parish and St. John the Baptist Parish. Those used by Law are identified on Figure 2.4-39 and tabulated on Table 2.4-13. More than one-half the wells identified in the above referenced table are either not in use or destroyed. Only four percent of the wells were used for domestic purposes at the time of the inventory.

←(DRN 01-464)

In defining the area to be addressed in the detailed well survey in the field, the following items were taken into consideration:

a) The thalweg of the Mississippi is 122 ft. below MSL at this reach of the stream, nullifying any connection between the reactor site and shallow wells in the Norco area.

b) All major industrial wells have already been addressed in the regional well survey.

c) The groundwater velocities in the shallow, local strata are extremely slow, and move away from the river in a south-southwesterly direction.

For these reasons, the more comprehensive survey was confined to an area defined by a one mile radius around the Reactor Building on the west bank of the Mississippi River. The boundaries of this detailed field study are illustrated on Figure 2.4-40 and parallel the results of the regional survey. There is little expanded use of groundwater and very little of what is used goes for domestic water supply. Results of this survey are tabulated on Table 2.4-14. In addition to conducting a well survey, information was obtained regarding water quality at and around the site. This information is presented on Table 2.4-15.

Most of the groundwater used in the site area is taken from the Norco aquifer and is mainly for industrial purposes. The closest area of concentrated pumpage of groundwater is the Norco well field, about three miles northeast of the site and on the opposite bank of the Mississippi River. The effect of this pumping center upon groundwater levels in both the Gramercy and Norco aquifers at the site is shown in Figures 2.4-34 and 2.4-35, respectively. The largest groundwater consumption in the region occurs in the New Orleans area, about 25 miles east of the site. The effect of this pumping center upon groundwater levels beneath the site is shown in Figure 2.4-37.

Local subsidence has resulted from the withdrawal of groundwater at Baton Rouge, Norco, and New Orleans. Subsidence related to groundwater is discussed in detail in Subsection 2.5.1.3. The zone of local subsidence at Norco is “bowl shaped” with the greatest subsidence (0.6 ft.) centered at the Norco well field. The level data along the Mississippi River indicate that the subsidence due to pumping at Norco is limited to a one mile radius of Norco. There has not been any local subsidence at the site itself as a result of groundwater withdrawals at Norco or at other pumping centers in the region. Subsidence at the site is not anticipated in the future because of the declining trend in groundwater usage at Norco and New Orleans.

2.4.13.2.3 Projected Future Use of Groundwater

The future use of groundwater in the site area appears to be limited by groundwater availability and quality rather than demand. According to a study by the Louisiana Department of Public Works[68] , total groundwater requirements for the St. James, St. John the Baptist, and St. Charles Parishes are expected to increase from 26 mgd for 1970 to 300 mgd by 2020 based on past trends (Figure 2.4-41). The actual development of sufficient groundwater supplies to meet this requirement, however, is expected to be small.

→(LBDCR 15-026, R309)

The Norco aquifer is the principal aquifer in the area and is mainly used for industrial purposes at Norco. The development potential of the Norco aquifer is large, but is limited by water quality considerations. Heavy pumping in the Norco area and hydraulic connections with the Gramercy aquifer have resulted in, and will cause further, salt water intrusion into the Norco aquifer.[62] As a result of decreased usage of groundwater at the Norco refinery since 1965, groundwater levels in the Norco aquifer have been gradually rising. Further discussion of the development potential of the Norco aquifer is in Subsection 2.4.13.2.1.

←(LBDCR 15-026, R309)

Groundwater development of the other aquifers in the site vicinity is restricted by their limited extent and water quality considerations. Point bars in the site area are not important aquifers because of limited extent; the water is very hard and high in iron content. The shallow aquifers are of limited extent, have low permeability, and poor water quality restricting their utility in the site area.

The Gramercy aquifer is generally too high in magnesium and calcium[65] for use as a water supply in the area.[62,68] Increased pumpage of the Norco aquifer would increase vertical groundwater movement from the Gramercy into the Norco aquifer. This, in turn, would cause further encroachment of the fresh-salt water interface towards the region. Much of the Norco aquifer south of Norco that now contains salty water will eventually be flushed if pumping increases. The initial flushing will be displacement by salty water moving in from

the Gramercy aquifer and then by fresh water from point bar aquifers connected to the Gramercy.

→(LBDCR 15-026, R309)
The potential groundwater development of the Gonzales-New Orleans aquifer in the site area is limited primarily by water quality tolerances. While this aquifer is used extensively in the New Orleans area, large scale developments in the site area are not expected because of the advantages of a higher transmissivity in the overlying Norco aquifer, potential fresh salt water interface problems, and apparent lack of a significant local source of recharge. Regional water level declines in the Gonzales-New Orleans aquifer can be expected in the future due to projected higher demand at New Orleans, but this should have little effect on future requirements near the Waterford site.

In summary, it is believed that any large increase in groundwater withdrawal in the site area will result in a decrease in water quality, making the water unsuitable for many of its present and projected uses without costly treatment. Therefore, it is assumed that groundwater withdrawal in the site area will not increase significantly beyond the present rate, and surface water will be used to a greater extent to satisfy much of the expected groundwater demand.

2.4.13.3 Accident Effects

Operating a nuclear plant at this site will not adversely effect either the local or regional groundwater resources, even in the unlikely event that a radioactive liquid spill should occur.

It is important to note that the groundwater regime is locally protected from spillage from point sources at the Reactor Building or the Reactor Auxiliary Building by the Zone 1 and Zone 2 aquicludes and by the gradients within the various strata themselves. Piezometric monitoring from June 1972 to present indicates that shallow groundwater movement is away from the Mississippi River at all stages of flow.

Furthermore, the highly conservative accident hypothesis described herein postulates an instantaneous release into the Zone 3 aquifer. Although the average piezometer measurements for Zone 3 at the reactor site fluctuate between El. +9.2 ft. MSL (corresponding to high water deviation of +17.8 ft. MSL in the Mississippi River) and El. +3.4 ft. (corresponding to low water deviation of +4.9 ft. in the Mississippi River). detailed subsurface investigations have already indicated that the material sandwiched between the base of the Reactor Building RAB and the Zone 3 aquifer exhibits a vertical permeability of 10^{-8} cm/sec. and it would not be possible under the present condition for material released from a point source at the base of the Reactor Building to enter the Zone 3 aquifer, (neglecting the vitiating effects of ion exchange in the clay materials. etc). Elaborate measures have been taken to insure that borings and other preoperational features are plugged and cannot cause a filtering effect through Zone 2 to the aquifer.

Calculations have been made of groundwater velocity in the El. -89 ft. MSL sand layer at various stages of the Mississippi River. The most conservative case (shown in Figure 2.4-42) is that where the river is at high level. Based on laboratory test results, it is assumed that the El. -89 ft. MSL sand layer has a permeability of 3×10^{-5} cm/sec. or 31 ft./yr. Using Darcy's Equation, a gradient of 0.008 (based on corresponding piezometric
←(LBDCR 15-026, R309)

→(LBDCR 15-026, R309)
levels in the sand layer) and a distance of 1180 ft. from the Mississippi River to the center of the reactor, a groundwater velocity of 0.234 ft./yr is obtained. Thus, in the unlikely event that radioactive material were to directly enter the El. -89 ft. MSL sand layer (after having instantaneously traversed 30 ft. of highly impermeable clay), it would still take an additional 1000 years to travel 234 ft. from the Reactor Building in a southwest direction from the Mississippi River.

There is evidence that the recent material (Zone 1) is discontinuous. At all stages of the Mississippi River, piezometric monitoring in this zone over a period of several months also indicates that the piezometrlc gradient is away from the river at all stages of flow (See Figure 2.4-43). However, it is instructive to make calculations assuming both continuous material and a direct and instantaneous entrance of radioactive material into the recent material at high river stages. Using Darcys Equation, a gradient of 0.009, a permeability of 1.5 x 106 cm/sec, or 1.5 ft./yr and a distance of 1470 ft., a groundwater velocity of 0.009 ft./yr is obtained with flow from the Reactor Building in a general direction away from the river. Thus, if radioactive material were to enter the recent material, it would take 100 years to go one foot. Piezometric readings in the recent material at the reactor site (and taken during the preconstruction period) fluctuate between +5.9 ft. (corresponding to a high water elevation of +17.8 ft. in the river) and +4.0 ft. (corresponding to a low water elevation of +49 ft. in the river).
←(LBDCR 15-026, R309)

It is therefore concluded from this analysis that there is no danger of contaminating the local or regional groundwater regime through the introduction of radioactive materials from accidental spillage.

2.4.13.4 Monitoring or Safeguard Requirements

In view of the use of other than groundwater resources for safety related purposes in connection with the operation of Waterford 3, the minimal utilization of groundwater resources (or expansion plans) in the immediate vicinity of the site, and the comparative isolation of the plant itself from the local groundwater regime in the event of radioactive spillage; there is no need to establish monitoring or safeguard requirements with respect to groundwater beyond the construction phase.

2.4.13.5 Design Basis for Subsurface Hydrostatic Loading

This topic is addressed in detail in Subsection 2.5.4, "Stability of Subsurface Materials."

2.4.14 TECHNICAL SPECIFICATIONS AND EMERGENCY OPERATION REQUIREMENTS

The Technical Specifications require monitoring potential flooding and securing all openings below EL. +30.0 ft. MSL when site flooding becomes a possibility.

SECTION 2.4: REFERENCES

1) Public Affairs Office, Mississippi River Commission and U. S. Army Engineer Division, Lower Mississippi Valley, March 1976, Flood Control in the Lower Mississippi River Valley.

2) Lower Mississippi Valley Division and NRC, in cooperation with North Central Division, Missouri River Division, Southwestern Division, and Ohio River Division, Mississippi River and Tributaries - Post-Flood Report 1973, Jan. 1974.

3) Department of the Army, Lower Mississippi Valley Division, Water Resources Development in Louisiana, 1975.

4) Everett, Duane, 1971, "Hydrologic and Quality Characteristics of the Lower Mississippi River", Technical Report No. 5, Louisiana Department of Public Works.

5) "Water Situation in Louisiana", September 1976 Newsletter, U. S. Geological Survey, Water Resources Division.

6) Surface Water Resources and Requirements for Louisiana 1970-2020, Comprehensive Water and Related Land Resources Study. Jan. 1974, Louisiana Department of Public Works.

7) Dial, Don C., Personal Communication concerning Municipal Water Pumpage for Downstream Users in 1975. U. S. Geological Survey, Water Resources Division, April 4. 1977.

8) Dial, Don C., 1970, "Public Water Supplies in Louisiana", Basic Records Report No. 3, Louisiana Department of Public Works.

9) Snider, J. C., Pumpage of Water in Louisiana, 1960", Louisiana Dept. of Public Works and Louisiana Geological Survey, May 1961.

10) Bieber, P. P., and Forbes, Jr., M. J .," Pumpage of Water in Louisiana, 1966", Dept. of Conservation - Louisiana Geological Survey, and Louisiana Dept. of Public Works, Aug. 1966.

11) Dial , Don C.," Pumpage of Water in Louisiana. 1970", Dept of Conservation - Louisiana Geological Survey and Louisiana Dept. of Public Works, July 1970.

12) Cardell . Rollo. Long. "Basic Groundwater Data for the Mississippi River Parishes South of Baton Rouge, Louisiana", Louisiana Department of Public Works. 1963.

13) USGS and NOAA, Geological Survey Professional Paper 937 - The 1973 Mississippi River Basin Flood: Compilation and Analyses of Meteorologic, Streamflow. and Sediment Data.

14) Neely. Braxtel, L. Jr.. U. S. Geological Survey, 1976, "Floods in Louisiana, Magnitude and Frequency". Third Edition. Published by Louisiana Department of Highways, Baton Rouge. La.

SECTION 2.4 REFERENCES (Cont'd.)

15) U. S. Nuclear Regulatory Commission, Regulatory Guide 1.59, Design Basis Floods for Nuclear Power Plants, Revision 2 (1976).

16) Riedel, J. T., J. F Appleby and R. W. Schloemer, 1956, "Seasonal Variation of the Probable Maximum Precipitation East of the 105th Meridian for Areas from 10 to 1000 Square Miles and Durations of 6, 12, 24, and 48 hours" U. S. Department of Commerce, Weather Bureau, Hydrometeorological Report No. 33.

17) U. S. Army Corps of Engineers, 1965, "Standard Project Flood Determinations" Civil Engineer Bulletin No. 52-8, EM 1110-2-1411.

18) U. S. Nuclear Regulatory Commission, Safety Evaluation Report, Waterford SES Unit 3, Dec. 1972.

19) Meyers, Vance A., U. S. Army Corps of Engineers and Weather Bureau (NOAA), December 1959, Hydrometeorological Report No. 35, Meteorology of Hypothetical Flood Sequences in the Mississippi River Basin.

20) Mississippi River and Tributaries Project, House Document No. 308, Vol. II,2 1964.

21) National Engineering Science Company, September 6. 1966, Storm Surge Effects in the Lower Mississippi River, Study B Contract No. DA-16-O47CIVENG 66-136.

22) Hydrologic Engineering Center, U. S. Army Corps of Engineers, Davis, Calif., Oct. 1973, HEC-II Water Surface Profile Computer Program, Users Manual.

23) U. S. Army Engineer District, New Orleans, June 1976, Mississippi River Hydrographic Survey, 1973-1975.

24) New Orleans District, U. S. Army Corps of Engineers, August 1974, Flood of 1973, Post-Flood Report Volume II.

25) Department of the Army, U. S. Army Engineer District, New Orleans. La., August 1976, Flood of 1975. Post-Flood Report.

26) Department of the Army New Orleans District. Corps of Engineers, Memo dated 31 March 1977, Mississippi River Levees. Levee Profile - LaFourche Levee District (Below New Orleans), Orleans Levee District (West). LaFourche Levee District (above New Orleans), and Project Design Flood Flow Line.

27) New Orleans District. U. S. Army Corps of Engineers, August 1974, Flood of 1973. PostFlood Report Vol. I

28) Hershfield D. "Rainfall Frequency Atlas of the United States". Technical Paper No. 40, IJ.S. Weather Bureau. May, 1961.

SECTION 2.4 REFERENCES (Cont'd.)

29) U.S. Army Corps of Engineers, 1965, "Standard Project Flood Determinations", Civil Engineer Bulletin No. 52-8, EM 1110-2-1411.

30) Deleted

31) Kolb, C. R .,"Geologic Control of Sand Boils Along Mississippi River Levees"~ in Geomorphology and Engineering, D R Coates, ed, Dowden, Hutchinson and Ross Inc., 1976.

32) Henderson, F. N., Open Channel Flow, MacMillan Co. New York, 1966.

33) Stoker, J. J., Water Waves, John Wiley & Sons, New York, 1957.

34) Dressler, R. F., 1954, "Comparison of Theories and Experiments for the Hydraulic Dam-Break Wave". International Association of Hydrology, Assemblee Generale de Rome, 1954, Vol. III p.319.

35) American Society of Civil Engineers, 1975, Sedimentation Engineering, ASCE Manual No. 54.

36) Noble, C. C. 1976. "The Mississippi River Flood of 1973", in Geomorphology and Engineering, D.R. Coates, ed. Dowden, Hutchinson, and Ross, Inc.

37) International Commission on Large Dams, 1973, World Register of Dams. SECTION 2.4: REFERENCES (Cont ' d)

38) HUR 7-97, 1968 Interim Report-Meteorological Characteristics of the Probable Maximum Hurricane, Atlantic and Gulf Coasts of the United States, Weather Bureau Memorandum to U. S. Department of Commerce.

39) U. S. Army Engineer District, New Orleans, 1972: History of Hurricane Occurrences along Coastal Louisiana. New Orleans, August 1972.

40) Marinos, G. and Woodward, J.. 1968: "Estimation of Hurricane Surge Hydrographs", Journal of the Waterways and Harbors Division, ASCE, May 1968.

41) Bodine, B., 1971: Storm Surge on the Open Coast: Fundamentals and Simplified Prediction. T. M #35. U. S. Army Coastal Engineering Research Center, May 1971.

42) U. S. NRC Regulatory Guide No. 1.59, 1976, Appendix C: "Simplified Methods of Estimating Probable Maximum Surges."

43) PSAR. 1969. St. Rosalie Generating Station, Louisiana Power & Light Co.

44) Reid. R. and Bodine. B.. 1968: "Numerical Model for Storm Surges in Galveston Bay", Journal of the Waterways and Harbors Division, ASCE. February 1968.

SECTION 2.4 REFERENCES (Cont'd.)

45) Ippen, A., 1966: Estuary and Coastline Hydrodynamics, McGraw-Hill Inc., New York.

46) U. S. Army Coastal Engineering Research Center, 1973: Shore Protection Manual.

47) Van Dorn, W. G., 1965, "TSUNAMIS", Advances in Hydroscience, Volume 2, Academic Press Inc., New York and London.

48) Wiegel, Robert L., 1970, Earthquake Engineering, Prentice Hall Inc., Englewood Cliffs. N. J.

49) Palermo, Robert C., Personal Communications Concerning Icing Conditions for the Lower Mississippi River, Army Corps of Engineers, Vicksburg, Mississippi, April 13, 1977.

50) Vicroy, Clarence E., Hydrologist, National Weather Service, Lower Mississippi River Forecasting Center Slidell, La. April 13, 1977.

51) Cardwell, George, Hydrologist, U. S. Geological Survey, Baton Rouge, La., April 1,1977.

52) Thompson, B. W., Lt. Comm. Chief of Search and Rescue (Ice Officer) U. S. Coast Guard, 2nd District, St. Louis, Mo., May 1, 1977.

53) Mississippi River and Commission and U. S. Army Engineer Division. Lower Mississippi Valley, 1976, "Flood Control in the Lower Mississippi River Valley", March 1976.

54) Forbes, Max J., Personal Communication Concerning Frequency Analysis of Mississippi River Discharge at Tarbert Landing, U. S. Geological Survey, Baton Rouge, La. March 3, 1977.

55) Miller, John R., Personal Communication Concerning Frequency Analysis of Mississippi River Discharge at Tarbert Landing, U.S.G.S., Baton Rouge, La. March 3, 1977.

56) Martens. L. A., 1974. "Time of Travel of Solutes in Mississippi River from Baton Rouge to Pointe a La Hache. Louisiana". Louisiana Department of Public Works, Water Resources Technical Report No. 9.

57) Everett. Duane E., 1971. "Hydrologic and Quality Characteristics of the Lower Mississippi River". Louisiana Department of Public Works, Technical Report No. 5.

58) Cardwell . G. T., Forbes, M. J., Jr., and Gaydos, M. W., 1967. Water Resources of the Lake Ponchartrain Area. Louisiana: U. S. Geological Survey. Water Resources Bulletin Number 12.

59) Cardwell. G. T.. and Dial, Don C.. 1974 and 1977. Personal Communication, U.S. Geological Survey. Water Resources Division, Baton Rouge. Louisiana.

SECTION 2.4 REFERENCES (Cont'd.)

60) Caughman, Bill, 1977, Personal Communication, Shell Oil Company, Norco Refinery.

61) Eddards, M. L., Kister, L. R., and Scarcia, Glenn, 1955, Water Resources of the New Orleans Area, Louisiana: U. S. Geological Survey Circular 374.

62) Hosman, R. L., 1972, Groundwater Resources of the Norca Area, Louisiana: U. S. Geological Survey, Water Resources Bulletin Number 18.

63) Kolb, C. R., 1962, Distribution and Engineering Significance of Sediments Bordering the Mississippi from Donaldsonville to the Gulf: Louisiana State University, Ph.D.. 1962, Geology.

64) Long, Richard A., 1965, Groundwater in the Geismar-Gonzales Area, Ascension Parish. Louisiana:U. S. Geological Survey, Water Resources Bulletin Number 7.

65) Rollo, J. R., 1966, Groundwater Resources of the Greater New Orleans Area, Louisiana: U.S. Geological Survey, Water Resources Bulletin Number 9.

66) U. S. Geological Survey, Water Resources Division, Basic Records, Report Number 7.

67) U. S. Geological Survey files, Water Resources Division, 1974 and 1977, Baton Rouge, Louisiana.

68) Louisiana Dept. of Public Works, October 1971 "Groundwater Resources and Requirements for Louisiana, 1970 - 2020" Comprehensive Water and Related Land Resources Study Series II, Volume II.

TABLE 2.4-1

MUNICIPAL WATER INTAKE DOWNSTREAM OF WATERFORD 3 DISCHARGE (RIVER MILE 129.6)

AREA OR WATER DISTRICT	RIVER MILE	1975 WATER PUMPAGE DATA MGD
St. Charles Water Works District No. 1	125.1	2.50
St. Charles Water Works District No. 2	120.6	2.50
East Jefferson Water Works District No. 1	105.4	36.86
New Orleans (Carrolton)	104.7	122.53
City of Westwego Water District	101.5	3.17
Jefferson Water Works District No. 2	99.1	15.20
City of Gretna Water District	96.7	3.21
New Orleans (Algiers Plant)	95.8	8.74
St Bernard Water Works District No.	87.9	9.00
Dalcour Water Works District	80.9	.33
Belle Chasse Water Works District	75.8	2.16
Pointe-a-la-Hache Water District	49.2	.23
Port Sulphur	39.4	1.08
Buras Water Works District, Empire	29.9	1.58
Boothville-Venice Water Works	18.6	1.01

Ref. USGS, Water Resources Division, Baton Rouge, LA 1977
(These pumpage figures are preliminary unpublished data and are subject to revision.)

TABLE 2.4-2

FLOOD-CREST ELEVATIONS NEAR THE WATERFORD SITE

Location	Miles Above Head of Passes, La.	1973 FLOOD DATA Date	Flood Peak (cfs)	Elevation in Ft., MSL	OTHER HISTORICAL Date	Flood Peak (cfs)	Elevation in Ft., MSL
Tarbert Landing, Miss.	306.3	May 13,14,15	-	59.30	Feb 19,1937	1,977,000	54.61[a]
		May 16	1,498,000	59.20			
Red River Landing, La.	302.4	May 13	-	58.22	May 14-17,1927	1,779,000[b]	60.94
Morganza, La.	275.4	May 13	-	53.20	-		-
Bayou Sara, La.	265.4	May 14,15	-	50.66	May 15,1927	-	55.46
Baton Rouge, La.	228.4	May 10	-	41.58	May 15,1927	-	47.28
Donaldsonville, La.	175.4	Apr.9	-	31.11	May 15,1927	-	36.01
College Point, La.	157.4	Apr.8	-	27.82	May 15, 1927	-	32.32
Reserve, La.	138.7	Apr.8	-	24.50	Jun 11, 1929	-	26.00
Bonnet Carre A (at Montz) La.	129.2	Apr.8	-	22.70	-	-	-
Bonnet Carre (Tower on left Bank) La.	128.0	Apr.8	-	22.57	Jun 10,1929	-	23.79
Bonnet Carre B (at Norco) La.	126.4	May 16	-	21.20	-	-	-
New Orleans (Carrollton) La.	102.8	Apr.7	-	18.47	Apr. 25, 1922	-	21.27
		Apr.15	1,257,000	-			

[a] Red River Landing Stage

[b] If discharge had been confined between levees

TABLE 2.4-3

MAXIMUM DISCHARGE FOR 1973 FLOOD (2)

River and Location	Maximum Discharge (cfs) Observed	Unregulated*
Tributaries:		
Missouri River, Herman, Missouri	500,000	560,000
Ohio River, L&D No. 51	570,000	610,000
Ohio River, L&D No. 52	920,000	1,070,000
White River, Clarendon, Arkansas	191,200	220,000
Arkansas River, Little Rock Arkansas	329,000	490,000
Yazoo River, Below Steele Bayou	75,000	130,000
Ouachita River, Monroe, Louisiana	87,900	87,900
Red River, Alexandria, Louisiana	142,000	167,000
Mississippi River:		
Alton, Illinois	535,000	560,000
St. Louis, Missouri	852,000	910,000
Cairo, Illinois	1,519,000	1,7841000
Memphis, Tennessee	1,633,000	1,883,000
Arkansas City, Arkansas	1,879,000	2,050,000
Vicksburg, Mississippi	1,962,000	2,102,000
Natchez, Mississippi	2,017,000	2,150,000
Latitude of Red River Landing	2,261,000	2,391,000

*Estimated maximum discharge with no reservoirs in the Mississippi River Basin.

TABLE 2.4-4

ORIGINAL AND ADJUSTED PROJECT DESIGN[2]

FLOOD FLOW LINES

Location	Original Flow Line, Ft MSL	1973 Adjusted Flow Line, Ft MSL	Change Ft.
Mhoon Landing, Arkansas	213.3	213.3	0
Helena, Arkansas	204.3	204.8	+0.5
Arkansas City, Arkansas	154.1	158.6	+4.5
Vicksburg, Mississippi	105.4	110.9	+5.5
Natchez, Mississippi	80.0	84.5	+4.5
Red River Landing, Louisiana	61.0	65.0	+4.0
Baton Rouge, Louisiana	46.4	47.4	+1.0
Donaldsonville, Louisiana	33.6	33.6	0

TABLE 2.4-5 (Sheet 1 of 2)

STREAMFLOW IN THE MISSISSIPPI RIVER *
1900-1976

Discharge (in 1000 cfs)

Year	Maximum	Minimum	Mean
1900	796	157	434
1901	822	104	377
1902	861	198	461
1903	1206	116	639
1904	1018	119	465
1905	918	165	576
1906	1116	253	592
1907	1275	198	676
1908	1218	138	667
1909	1163	157	581
1910	853	130	473
1911	1007	174	459
1912	1499	198	646
1913	1272	167	584
1914	903	137	409
1915	934	298	653
1916	1327	157	641
1917	1218	110	510
1918	727	110	400
1919	960	154	602
1920	1223	181	657
1921	992	156	527
1922	1437	133	566
1923	1126	226	590
1924	928	154	549
1925	656	104	368
1926	813	143	477
1927	1779	173	867
1928	1035	236	601
1929	1301	163	643
1930	911	125	419
1931	672	119	283
1932	1244	158	516
1933	1076	130	522
1934	720	130	292
1935	1087	112	574
1936	973	92	346

TABLE 2.4-5 (Sheet 2 of 2)

STREAMFLOW IN THE MISSISSIPPI RIVER *
1900-1976

Discharge (in 1000 cfs)

Year	Maximum	Minimum	Mean
1937	1467	128	514
1938	1062	131	511
1939	1124	75	445
1940	872	93	313
1941	749	146	376
1942	973	242	499
1943	1280	133	520
1944	1282	125	475
1945	1520	179	683
1946	1085	145	509
1947	898	114	426
1948	959	126	448
1949	1208	176	555
1950	1458	194	696
1951	986	221	625
1952	1011	107	466
1953	852	100	373
1954	583	121	262
1955	1022	120	363
1956	894	99	332
1957	994	180	548
1958	984	157	482
1959	765	130	382
1960	826	148	409
1961	1107	183	514
1962	1081	151	475
1963	881	123	268
1964	1015	119	366
1965	936	168	417
1966	1154	155	372
1967	803	180	384
1968	857	160	434
1969	1064	186	460
1970	980	178	451
1971	1036	174	338
1972	938	218	480
1973	1498	204	721
1974	1174	187	586
1975	1216	230	563
1976	721	158	364**

* 1900-1963 Discharge at Red River Landing, Louisiana and
1964-1976 Discharges at Tarbert Landing, Mississippi

** Army Corps of Engineers Data Marked: PRELIMINARY - SUBJECT TO REVISION

TABLE 2.4-6 Revision 11 (05/01)

→ (DRN 99-2493)

TABLE 2.4-6 HAS BEEN INTENTIONALLY DELETED.

← (DRN 99-2493)

TABLE 2.4-6a Revision 11 (05/01)

(DRN 99-2493)

TABLE 2.4-6a HAS BEEN INTENTIONALLY DELETED.

(DRN 99-2493)

TABLE 2.4-6b Revision 11 (05/01)

→ (DRN 99-2493)

TABLE 2.4-6b HAS BEEN INTENTIONALLY DELETED.

← (DRN 99-2493)

TABLE 2.4-6c Revision 11 (05/01)

→ (DRN 99-2493)

TABLE 2.4-6c HAS BEEN INTENTIONALLY DELETED.

← (DRN 99-2493)

TABLE 2.4-7

HYPOTHETICAL FLOOD PEAKS ON THE
MISSISSIPPI AND ATCHAFALAYA RIVERS

Flood Identification	Peak Flow at Latitude of Red River Landing (cfs)	
	Without Upstream Regulation	With Upstream Regulation
Hypothetical Winter Flood (No. 58A)	3,320,000	3,030,000
Hypothetical Early Spring Flood (No. 56)	3,180,000	2,670,000
Hypothetical Late Spring Flood (No. 63)	2,730,000	2,480,000
Hypothetical Early Summer Flood (No. 52A)	2,250,000	1,900,000

TABLE 2.4-8 Revision 11-A (02/02)

COMPUTATIONS OF STATIC AND DYNAMIC HEAD FOLLOWING LEVEE FAILURE

Case 1 Yo = 13 ft

→(DRN 01-464)

y	$\sqrt{gy}$	v	$\frac{v^2}{2g}$	$y+\frac{v^2}{2g}$	Elevation
(Ft.)	(Ft/Sec.)	(Ft./Sec.)	(Ft.)	(Ft.)	(Ft.MSL)
2.6	9.2	22.7	8.0	10.6	24.6
3	9.8	19.6	6.4	9.4	23.4
4	11.4	18.3	5.4	9.4	23.4
5	12.7	15.6	3.8	8.8	22.8
6	13.9	13.2	2.7	8.7	22.7

←(DRN 01-464)

Case 2 Yo = 14 ft

2.8	9.5	23.5	8.6	11.4	25.4

Where:

v = Water velocity

g = Gravitational acceleration

y = Water depth at plant wall

y_o = River stage - 14.0

Elevation = 14.0 ft + y + v ^{2} /2g

TABLE 2.4-9

COMPUTATIONS OF STATIC AND DYNAMIC HEAD

RIVER STAGE = 30 Ft MSL

y_o = 12.5 ft.

y (ft)	v (ft/sec)	$v^2/2g$ (ft)	Elevation (ft MSL)
2.5	22.18	7.64	27.64
3.0	20.47	6.51	27.01
3.5	18.89	5.54	26.54
4.0	17.43	4.72	26.22
5.0	14.75	3.38	25.88
5.56	13.37	2.78	25.83

Where:

Elevation = 17.5 ft + y + $v^2/2g$
y = depth at NPIS
v = velocity of flow at depth y
g = gravitational acceleration

TABLE 2.4-10

PROBABLE MAXIMUM HURRICANE PARAMETERS
ZONE B -MILE 660 - NEW ORLEANS, LOUISIANA

Case	Radius to Max Winds (nautical miles)	Forward Speed (knots)	Central Pressure (in-Hg)	Peripheral Pressure (in-Hg)	Maximum Wind (mph)
1	7	4	26.90	31.26	138
2	14	4	26.90	31.26	138
3	30	4	26.90	31.26	138
4	7	11	26.90	31.26	143
5	14	11	26.90	31.26	143
6	30	11	26.90	31.26	143
7	7	28	26.90	31.26	153
8	14	28	26.90	31.26	153
9	30	28	26.90	31.26	153

TABLE 2.4-10A

COMPARISION OF HURRICANE SURGE COMPUTATIONS
FOR SOUTH PASS APPROACH

Central Pressure Index = 26.90 in. Hg
Peripheral Pressure = 31.26 in. Hg
Radius to Max Winds = 30.0 n mi

Case (from Table 2.4-10)	3	9
Forward Speeds, knots	4	28
Max Wind Speed, mph	138	153
Bottom Friction Facto	0.0001	0.0001
Wind Setup, ft.	9.15	8.89
Pressure Setup, ft.	2.92	3.20
Initial Rise, ft-	2.0	2.0
Astro Tide, ft.MSL	2.0	2.0
Total Surge, ft.MSL	16.06	16.09

TABLE 2.4-11

CONTINENTAL SHELF PROFILES FOR
OPEN COAST SURGE COMPUTATIONS

SOUTH PASS APPROACH		BARATARIA BAY APPROACH	
DISTANCE FROM SHORE (N.M.)	DEPTH (FEET)	DISTANCE FROM SHORE (N.M.)	DEPTH (FEET)
14.0	600.0	47.0	600.0
13.0	515.0	46.0	430.0
12.0	420.0	42.5	375.0
11.0	335.0	38.5	327.0
10.0	255.0	35.0	280.0
9.0	200.0	28.5	220.0
8.0	170.0	27.0	192.0
7.0	120.0	24.0	175.0
6.5	75.0	21.0	150.0
6.0	50.0	17.0	117.0
5.0	42.0	13.5	90.0
4.0	33.0	10.3	65.0
3.0	25.0	7.0	45.0
2.0	17.0	4.5	37.0
1.0	8.0	2.5	27.0
0.0	0.0	1.0	8.0
		0.0	0.0

TABLE 2.4-12

FREQUENCY ANALYSIS OF MISSISSIPPI RIVER DISCHARGE AT TARBERT LANDING*

FLOW (CFS)	DURATION (% TIME)
140,000	95
170,000	89.1
200,000	82.6
220,000	77.1
270,000	66.4
320,000	57.2
390,000	47.9
460,000	40.5
560,000	31.8
670,000	23.6
800,000	14.8
870,000	11.1
960,000	6.9
1,050,000	3.9
1,150,000	2.0

*Period of record 1930-1975 combines data from Red River Landing and Tarbert Landing

Ref. Unpublished preliminary data - subject for revision
USGS, Baton Rouge, 1977

TABLE 2.4-13 (Sheet 1 of 9)

WATER WELL AND TEST HOLE DATA FOR ST. CHARLES PARISH

EXPLANATION

Local Well No. - Consists of an abbreviation of St. Charles Parish followed by the number which represents the sequential order of inventory by the U. S. Geological Survey.

Location - Section, township, and range

Note: The information provided in this table does not represent all wells within St. Charles Parish, only those inventoried by the U.S. Geological Survey.

Use - The following symbols indicate usage:

- Ind. - Industrial
- X - Destroyed or abandoned
- U - Unused
- T - Test hole only
- S - Stock
- D - Domestic
- Irr - Irrigation
- Ob. - Observation

*Ob. - Observation well in which water level records are maintained by the U.S. Geological Survey.

Recorded Date - Date of inventory

Source of Data: Reference 2.4.13-10
2.4.13-9

TABLE 2.4-13 (Sheet 2 of 9)

WATER WELL AND TEST HOLE DATA FOR ST. CHARLES PARISH

EXPLANATION

LOCAL WELL NO.	LOCATION	OWNER OR USER	USE	TOTAL DEPTH	PROBABLE AQUIFER TAPPED	RECORDED DATE
SC-1	Sec.6, T.12S.,R.8E.	Shell Oil Company	X	420	Norco	1943
SC-2	Sec.6, T.12S.,R.gE-	Shell Oil Company	X	420	Norco	1943
SC-3	Sec.6, T.12S.,R.8E.	Shell Oil Company	X	420	Norco	1943
SC-4	Sec.6, T.12S.,R.8E-	Shell Oil Company	X	450	Norco	1943
SC-5	Sec.6, T.12S.,R.8E-	Shell Oil Company	X	444	Norco	1943
SC-6	Sec.6, T.12S.,R.8E-	Shell Oil Company	X	404	Norco	1943
SC-7	Sec.6, T.12S.,R.8E-	Shell Oil Company	X	448	Norco	1943
SC-8	Sec.6, T.12S.,R.8E-	Shell Oil Company	X	769	Gonzales - New Orleans	1943
SC-9	Sec.6, T.12S.,R.8E.	Shell Oil Company	*Ob.	808	Gonzales - New Orleans	1943
SC-10	Sec.6, T.12S.,R.8E.	Shell Oil Company	X	835	Gonzales - New Orleans	1943
SC-11	Sec.6, T.12S.,R.8E.	Shell Oil Company	X	841	Gonzales - New Orleans	1943
SC-12	Sec.6, T.12S.,R.8E.	Shell Oil Company	X	436	Norco	1943
SC-13	Sec.6, T.12S.,R.8E.	Shell Oil Company	X	457	Norco	1943
SC-14	Sec.6, T.12S.,R.8E-	Shell Oil Company	*Ob.	404	Norco	1943
SC-15	Sec.6, T.12S.,R.8E.	Shell Oil Company	Ind.	864	Gonzales - New Orleans	1943
SC-16	Sec.6, T.12S.,R.8E.	Shell Oil Company	Ind.	754	Gonzales - New Orleans	1943
SC-17	Sec.6, T.12S.,R.8E.	Shell Oil Company	Ind.	783	Gonzales - New Orleans	1943
SC-18	Sec.6, T.12S.,R.8E.	Shell Oil Company	Ind.	414	Norco	1943
SC-19	Sec.4, T.13S.,R.8E.	Pan - Am Refinery	X	420	Norco	1943
SC-20	Sec.4, T.13S.,R.8E	Pan - Am Refinery	Ind.	484	Norco	1943
SC-21	Sec.4, T.13S.,R.8E	Pan - Am Refinery	X	310	Norco	1943
SC-22	Sec.6, T.12S.,R.gE.	Henry Saizan	X	389	Norco	1960

TABLE 2.4-13 (Sheet 3 of 9)

WATER WELL AND TEST HOLE DATA FOR ST. CHARLES PARISH

EXPLANATION

LOCAL WELL NO.	LOCATION	OWNER OR USER	USE	TOTAL DEPTH	PROBABLE AQUIFER TAPPED	RECORDED DATE
SC-23	Sec.6, T.12S., R.8E.	W. N. Kugler, Jr.	U	385	Norco	1960
SC-24	Sec.40, T-12S., R.8E.	Intl. Tank Terminal	*Ob.	492	Norco	1960
SC-25	Sec.4, T-13S., R.8E.	Am. Oil Company	Ind.	476	Norco	1957
SC-26	Sec.35, T. 14S., R.22E.	Mrs. B. F. Clesi	X	363	Norco	1943
SC-27	Sec.39, T.14S., R.20E.	Mr. Folse	X	276	Gramercy	1943
SC-28	Sec.35, T.14S., R.20E.	Mr. J. Gros	Ob.	288	Gramercy	1957
SC-29	Sec.39, T.14S., R.20E.	0. J. Boyer	U	276	Gramercy	1957
SC-30	Sec.119,T.13S., R.20E.	Stanley Tinney	X	360	Norco	1957
SC-31	Sec.18, T.13S., R.20E.	Preston Madere	U	147	Shallow aquifer	1957
SC-32	Sec.18, T.13S., R.20E.	Preston Madere	U	117	Shallow aquifer	1957
SC-33	Sec.26, T-12S., R.20E.	Waterford Sugar Co-op,Inc.	X	350	Norco	1943
SC-34	Sec.26, T.12S., R.20E.	MulTican & Farwell	X	387	Norco	1943
SC-35	Sec.18, T.13S., R.20E.	St. Charles Parish Police Jury	X	260	Gramercy	1957
SC-36	Sec.60, T.12S., R.19E.	Tom Landeche	U	364	Norco	1943
SC-37	Sec.24, T.12S., R.20E.	Southern Dairy Prods.	X	500	Norco	1943
SC-38	Sec.23, T.12S., R.20E.	Southern Dairy Prods.	X	278	Gramercy	1943
SC-39	Sec.48, T.14S., R.20E.	Des Allemandes School	X	307	Gramercy	1943
SC-40	Sec.48, T.14S., R.20E.	Lakeside Fisheries	U	315	Norco	1943
SC-41	Sec.39, T.14S., R.20E.	Paradis School	U	285	Gramercy	1943
SC-42	Sec.38, T.13S., R.9E.	Dr. Mattingly	D	460	Norco	1943
SC-43	Sec.40, T.12S., R.9E.	Bob Landry	X	400	Norco	1943

TABLE 2.4-13 (Sheet 4 of 9)

WATER WELL AND TEST HOLE DATA FOR ST. CHARLES PARISH

EXPLANATION

LOCAL WELL NO.	LOCATION	OWNER OR USER	USE	TOTAL DEPTH	PROBABLE AQUIFER TAPPED	RECORDED DATE
SC-44	Sec.6, T.125 R.8E.	Shell Oil Company	X	424	Norco	1943
SC-45	Sec.6, T-125.R.8E.	Shell Oil Company	Ind.	402	Norco	1946
SC-46	Sec.6, T.12S.R.8E.	Shell Oil Company	Ind.	460	Norco	1946
SC-47	Sec.7,T.12S.,R.8E.	Pure Oil company	U	171	Shallow aquifer	1956
SC-48	Sec.4,T.13S.,R.8E.	Pan-Am Corporation	X	330	Gramercy	1948
SC-49	Sec.4,T.13S.,R.8E.	American Oil Co.	U	476	Norco	1948
SC-50	Sec.6,T.12S.,R.8E.	Shell Oil Company	Ind.	404	Norco	1948
SC-51	Sec.6,T.12S.,R.8E.	Shell Oil Company	Ind.	416	Norco	1948
SC-52	Sec.6,T.12S.,R.8E.	Shell Oil Company	Ind.	449	Norco	1948
SC-53	Sec.6,T.12S.,R.8E.	Shell Oil Company	X	453	Norco	1948
SC-54	Sec.6,T.12S.,R.8E.	Shell Oil Company	Ind.	464	Norco	1948
SC-55	Sec.6,T.12S.,R.8E.	Shell Oil Company	X	472	Norco	1948
SC-56	Sec.4,T-12S.,R.7E.	Emmel Perilloux	X	375	Norco	1948
SC-57	Sec.50, T-12S. R.8E.	La. Power & Light	X	380	Norco	1948
SC-58	Sec.7, T-12S., R.8E.	Shell Oil Company	X	364	Norco	1956
SC-59	Sec.7, T.12S., R.gE.	Humble Oil Company	Ind.	300	Norco	1948
SC-60	Sec.38, T-15S., R.20E.	Mrs. Andrew Hogan	U	258	Norco	1957
SC-61	Sec.39, T.14S., R.20E.	The Texas Company	U	298	Gramercy	1948
SC-62	Sec.11, T.14S., R.20E.	W.H. Talbot	*Ob.	273	Gramercy	1957
SC-63	Sec.14, T.13S., R.8E.	Sidney L. Hymel	X	475	Norco	1948
SC-64	Sec.6, T.12S., R.8E.	Shell Oil Company	X	875	Gonzales - New Orleans	1949
SC-65	Sec.6, T.12S., R.8E.	Shell Oil Company	X	882	Gonzales - New Orleans	1949

TABLE 2.4-13 (Sheet 5 of 9)

WATER WELL AND TEST HOLE DATA FOR ST. CHARLES PARISH

EXPLANATION

LOCAL WELL NO.	LOCATION	OWNER OR USER	USE	TOTAL DEPTH	PROBABLE AQUIFER TAPPED	RECORDED DATE
SC-66	Sec.39, T.IIS., R.8E.	Illinois Central R.R.	D	670	Gonzales - New Orleans	1949
SC-67	Sec.39, T.IIS., R.8E	Illinois Central R.R.	X	457	Norco	1949
SC-68	Sec.6, T.12S., R.gE.	Shell Oil Company	Ind.	463	Norco	1949
SC-69	Sec.6, T.12S., R.8E.	Shell Oil Company	Ind.	838	Gonzales - New Orleans	1950
SC-70	Sec.38, T.13S,. R.9E.	Charles Goodman	X	520	Norco	1960
SC-71	Sec.4, T.13S., R.8E.	American Oil Company	Ind.	489	Norco	1955
SC-72	Sec.6, T.12S., R.8E.	Shell Oil Company	Ind.	409	Norco	1956
SC-73	Sec.8, T.13S., R.20E.	Leon C. Vial, Jr.	S	262	Gramercy	1956
SC-74	Sec.42, T.13S., R20E.	A. N. Zimmer & Son	U	367	Norco	1956
SC-75	Sec.41, T.13S., R.20E.	A. N. Zimmer & Son	S	367	Norco	1956
SC-76	Sec.7, T.12S., R.8E.	Humble Oil Ref. Co.	U	379	Norco	1957
SC-77	Sec.7, T.12S., R.8E.	Humble Oil Ref. Co.	U	373	Norco	1957
SC-78	Sec.7, T.12S., R.8E.	Humble Oil Ref. Co.	U	---	---	1957
SC-79	Sec.7, T.12S., R.8E.	Huble Oil Ref. Co.	U	---	---	1957
SC-80	Sec.15, T.13S., R.21E.	Ambrose Champagne	*Ob.	256	Gramercy	1957
SC-81	Sec.65, T.13S., R.21E.	Pizzolato & Post	X	300	Gramercy	1957
SC-82	Sec.26, T.12S., R.20E.	La. Power & Light Co.	*Ob.	373	Norco	1957
SC-83	Sec.40, T.12S., R.9E.	Bob Landry	U	360	Norco	1957
SC-84	Sec.50, T.12S., R.BE.	La. Power & Light Co.	*Ob.	383	Norco	1957
SC-85	Sec.40, T.12S., R.9E.	California Company	Ind.	206	Gramercy	1957
SC-86	Sec.11, T.14S., R.20E.	W. H. Talbot	S	279	Gramercy	1957
SC-87	Sec.23, T.12S., R.20E.	So. Dairy Prods.	S	400	Norco	1957

TABLE 2.4-13 (Sheet 6 of 9)

WATER WELL AND TEST HOLE DATA FOR ST. CHARLES PARISH

EXPLANATION

LOCAL WELL NO.	LOCATION	OWNER OR USER	USE	TOTAL DEPTH	PROBABLE AQUIFER TAPPED	RECORDED DATE
SC-88	Sec.6, T.12S., R.gE.	Shell Oil Company	Ind.	480	Norco	1959
SC-89	Sec.7, T.12S., R.7E.	U.S. Army Corps of Engrs.	T	102	Shallow aquifer	1958
SC-90	Sec.7, T.12S., R.7E.	U.S. Army Corps of Engrs.	T	202	Gramercy	1958
SC-91	Sec.7, T.12S., R.7E.	U.S. Army Corps of Engrs.	T	102	Shallow aquifer	1958
SC-92	Sec.26, T.12S., R.20E	U.S. Army Corps of Engrs.	T	102	Shallow aquifer	1959
SC-93	Sec.70, T.12S., R.19E.	U.S. Army Corps of Engrs.	T	202	Gramercy	1959
SC-94	Sec.60, T.12S., R.19E.	Landeche Bros. Sugar Plantation	T	47	Shallow aquifer	1959
SC-95	Sec.62, T.12S., R.19E.	Landeche Bros. Sugar Plantation	T	72	Shallow aquifer	1959
SC-96	Sec.7, T.12S., R.8E.	Shell Oil Companyy	Ind.	370	Norco	1959
SC-97	Sec.27, T.13S., R.20E.	Dr. P. E. Landeche	T	---	---	1960
SC-98	Sec.27, T.13S., R.20E.	Dr. P. E. Landeche	T	67	Shallow aquifer	1960
SC-99	Sec.41, T.13S., R.9E.	George Francis	S	350	Norco	1960
SC-100	Sec.40, T.13S., R.9E.	Cities Service Oil Co.	T	97	Shallow aquifer	1960
SC-101	Sec.40, T.13S., R.9E.	Cities Service Oil Co.	T	82	Shallow aquifer	1960
SC-102	Sec.40, T.13S., R.9E.	U. S. Geol. Surv.	*Ob.	63	Shallow aquifer	1960
SC-103	Sec.12, T.12S., R.8E.	St. Charles Parish	T	62	Shallow aquifer	1960
SC-104	Sec.16, T.12S., R.20E.	A. N. Zimmer & Sons	U	396	Norco	1960
SC-105	Sec.32, T.14S., R.20E.	Texas Company	U	---	---	1960
SC-106	Sec.39, T.14S., R.20E.	Texas Company	U	372	Norco	1960
SC-107	Sec.47, T.14S., R.20E.	Dufrene Packing Co.	U	196	Gramercy	1960
SC-108	Sec.1, T.13S., R.22E.	T. B. Sellers	S	200	Gramercy	1960

TABLE 2.4-13 (Sheet 7 of 9)

WATER WELL AND TEST HOLE DATA FOR ST. CHARLES PARISH

EXPLANATION

LOCAL WELL NO.	LOCATION	OWNER OR USER	USE	TOTAL DEPTH	PROBABLE AQUIFER TAPPED	RECORDED DATE
SC-109	Sec.15, T-12S., R.8E	U.S. Geol. Surv.	T	82	Shallow aquifer	1960
SC-110	Sec.35, T.13S., R.21E.	John M. Walton, Jr.	U	80	Shallow aquifer	1960
SC-111	Sec.35, T.13S., R.21E.	Julius Sellers	T	100	Shallow aquifer	1960
SC-112	Sec.1, T-13S., R.22E.	T. B. Sellers	U	80	Shallow aquifer	1960
SC-113	Sec.8, T.13S., R.20E.	Mrs. L. C. Vial, Jr.	X	285	Gramercy	1960
SC-114	Sec.22, T.13S., R.20E.	D. A. Keller Estate	U	300-400	Norco	1960
SC-115	Sec.23, T.13S., R.20E.	L. M. Granier	U	300-400	Norco	1960
SC-116	Sec.1, T.13S., R.20E.	E. A. Dufresne, Sr.	U	---	---	1960
SC-117	Sec.39, T.14S., R.20E.	Humble Oil & Ref. Co.	U	310	Gramercy	1960
SC-118	Sec.47, T.13S., R.20E.	Dufrene Packing CO.	U	351	Gramercy	1960
SC-119	Sec.16, T.13S., R.21E.	Monsanto Chem. Co.	X	280	Gramercy	1960
SC-120	Sec.14, T.13S., R.21E.	St. Charles Parish School Board	X	270	Gramercy	1960
SC-121	Sec.6, T.12S., R.8E.	Shell Oil Company	T	770	Gonzales - New Orleans	1961
SC-122	Sec.6, T.12S., R.8E.	Shell Oil Company	T	870	Gonzales - New Orleans	1961
SC-123	Sec.6, T.12S., R.8E.	Shell Oil Company	T	845	Gonzales - New Orleans	1961
SC-124	Sec.21, T.12S., R.8E.	Shell Oil Company	T	869	Gonzales - New Orleans	1961
SC-125	Sec.6, T.12s., R.8E.	Shell Oil Company	T	857	Gonzales - New Orleans	1961
SC-126	Sec.6, T.12S., R.gE.	Shell Oil Company	T	850	Gonzales - New Orleans	1961
SC-127	Sec.6. T.12S., R.8E.	St. Charles Parish School Board	X	257	Gramercy	1961
SC-128	Sec.7, T.12S., R.8E.	Pure Oil Company	U	690	Gonzales - New Orleans	1961
SC-129	Sec.44, T.IIS., R.gE.	Woodrow Dufrene	U	325	Norco	1961

TABLE 2.4-13 (Sheet 8 of 9)

WATER WELL AND TEST HOLE DATA FOR ST. CHARLES PARISH

EXPLANATION

LOCAL WELL NO.	LOCATION	OWNER OR USER	USE	TOTAL DEPTH	PROBABLE AQUIFER TAPPED	RECORDED DATE
SC-130	Sec.6, T.12S., R.8E.	Mrs. A. Wetzka	U	380	Norco	1961
SC-131	Sec.11, T.12S., R.7E.	La. Power & Light	X	315	Norco	1961
SC-132	Sec.1, T.13S., R.8E.	A. W. Brown	U	420	Norco	1961
SC-133	Sec.1, T-13S., R.8E.	A. W. Brown	Irr.	700	Gonzales - New Orleans	1961
SC-134	Sec.15, T-13S., R.21E.	Rudolph Patterson	X	435	Norco	1961
SC-135	Sec.39, T.14S., R.20E.	Texaco Inc.	X	306	Norco	1961
SC-136	Sec.38, T.14S., R.20E.	Texaco Inc.	U	---	---	1961
SC-137	Sec.7, T.14S., R.20E.	Texaco Inc.	X	331	Gramercy	1961
SC-138	Sec.39, T-14S., R.20E.	Texaco Inc.	U	295	Gramercy	1961
SC-139	Sec.39, T.14S., R.20E.	Texaco Inc.	X	296	Gramercy	1961
SC-140	Sec.39, T-14S., R.20E.	Texaco Inc.	Ind.	286	Gramercy	1961
SC-141	Sec.39, T.14S., R.20E.	Alvin J. Egle	U	269	Gramercy	1961
SC-142	Sec.1, T.13S., R.20E.	Dr C Walter Hattingly	U	312	Gramercy	1961
SC-143	Sec.6, T-13S., R.20E.	Michael H. Brown	S	450	Norco	1961
SC-144	Sec.5, T.13S., R.20E.	Dr. Lawrence J O'Neil	X	315	Gramercy	1961
SC-145	Sec.119, T.13S. R.20E.	N. F. Shaak	Irr.	350	Gramercy	1961
SC-146	Sec.52, T.12S., R.19E.	Estate of G Montgomery	S	400	Norco	1961
SC-147	Sec.18, T.13S., R.20E.	St. Charles Parish Police Jury	X	350	Norco	1961
SC-148	Sec.18, T.13S., R.20E.	St. Charles Parish Police Jury	X	200	Gramercy	1961
SC-149	Sec.53, T.12S., R.19E.	St. Charles Parish School Board	X	407	Norco	1961

TABLE 2.4-13 (Sheet 9 of 9)

WATER WELL AND TEST HOLE DATA FOR ST. CHARLES PARISH

EXPLANATION

LOCAL WELL NO.	LOCATION	OWNER OR USER	USE	TOTAL DEPTH	PROBABLE AQUIFER TAPPED	RECORDED DATE
SC-150	Sec.4, T.13S.R.8E.	American Oil Company	X	500	Norco	1962
SC-151	Sec.6, T.12S.R.8E.	Shell Oil company	Ind.	1794	---	1962
SC-152	Sec.3, T.12S.R.8E.	Shell Chemical Co.	Ind.	1900	---	1962
SC-153	Sec.10, T.12S., R.7E.	La. Power & Light	T	125	Shallow aquifer	1962
SC-154	Sec.7, T-12S., R.7E.	U.S. Army Corps of Engrs.	T	142	Shallow aquifer	1962
SC-155	Sec.5, T.13S., R.20E.	U.S. Army Corps of Engrs.	T	110	Shallow aquifer	1962
SC-156	Sec.1, T-13S., R.8E.	U.S. Army Corps of Engrs.	T	154	Shallow aquifer	1962
SC-157	Sec.30, T-13S., R.21E.	U.S. Army Corps of Engrs.	T	175	Shallow aquifer	1962
SC-158	Sec.3, T.12S., R.8E.	Shell Chemical Company	Ind.	1830	---	1962
SC-159	Sec.23, T.12S., R.20E.	Occidental Chem. Co.	Ind.	440	Norco	
SC-160	Sec.22, T.13S., R.20E.	Mr. Theo Keller	U	289	Gramercy	1967
SC-161	Sec.16, T.13S., R.20E.	Union Carbide Corp.	D	283	Gramercy	1968
SC-162	Sec.16, T.13S., R.20E.	Union Carbide Corp.	D	378	Norco	1968
SC-163	Sec.16, T.13S., R.20E.	Union Carbide Corp.	D	378	Norco	1968
SC-164	Sec.17, T.13S., R.20E.	Argus Chem. Corp.	Ind.	410	Norco	1971

TABLE 2.4-14

LOCAL DETAILED WELL SURVEY

LOCAL WELL NO.	LOCATION	OWNER OR USER	USE+	DEPTH	PIEZOMETRIC LEVEL	DATE	PROBABLE AQUIFER TAPPED	DATE DRILLED
SC-33	Sec 26, T125,R20E	Waterford Suger	X	350	-	-	Norco	1943
SC-34	Sec 26, T125,R20E	Millican & Farwell	X	387	-	-	Norco	1943
SC-36	Sec 60, T125,R19E	Tom Landeche	U	364	-	-	Norco	1943
SC-37	Sec 24, T125,R20E	Southern Dairy Products	X	500	-	-	Norco	1943
SC-38	Sec 23, T125,R20E	Southern Dairy Products	X	278	-	-	Gramercy	1943
SC-74	Sec 42, T135,R20E	A.N. Zimmer & Sons	U	367	-	-	Norco	1956
SC-75	Sec 41, T135,R20E	A.N. Zimmer & Sons	S	367	-	-	Norco	1956
SC-82	Sec 26, T125,R20E	La. Power & Light Co.	Ob	373	21.4	June 1977	Norco	1957
SC-87	Sec 23, T125,R20E	Southern Dairy Products	S	400	-	-	Norco	1957
SC-92	Sec 26, T125,R20E	U.S. Army Corps of Engineers	T	102	-	-	Shallow	1959
SC-93	Sec 70, T125,R19E	U.S. Army Corps of Engineers	T	202	-	-	Gramercy	1959
SC-94	Sec 60, T125,R19E	Landeche Brothers Sugar Plantation	T	47	-	-	Shallow	1959
SC-95	Sec 62, T125,R19E	Landeche Brothers Sugar Plantation	T	72	-	-	Shallow	1959
SC-104	Sec 16, T125,R20E	A.N. Zimmer & Sons	U	396	-	-	Norco	1960
SC-131	Sec 11, T125,R7E	La. Power & Light Co.	X	315	-	-	Norco	1961
SC-146	Sec 52, T125,R19E	Estate of G. Montgomery	S	400	-	-	Norco	1961
SC-149	Sec 53, T125,R19E	St. Charles Parish School Board	X	407	-	-	Norco	1961
SC-159	Sec 23, T125,R20E	Occidental Chem. Co.	Ind.	440	-	-	Norco	-
*	Sec 26, T125,R20E	Argus Chem. Co.	Ind.	466	18	Aug. 1976	Norco	1976
*	Sec 26, T125,R20E	Beker Industries Corp.	Ind.	441	-	-	Norco	1964

Note:

* Not inventoried by U.S. Geological Survey
+ Same as PSAR inventory done by Law Engineering. See Table 2.4-13 for definitions

TABLE 2.4-15

GROUNDWATER QUALITY AT SITE

Well No.	SC-34	SC-36	SC-82	Arkus	P23	P31	Site Dewatering System	Mississippi River (at Luling Ferry) Max.	Min.	Ave.
Date of Log	1943	1943	1957	1964	1977	1977	1977	1968	1968	1968
Sio_2								7.9	0.8	4.9
Fe			.59	1.8				.26	0	.08
Ca				12.3				52	29	30
Mg				6.5				14	7.1	10.5
Na				337.4				40	15	23
K								5.4	1.2	3.2
CO_3				24						
HCO_3	570	548		541.7				179	89	123
SO4	1	1		1.4				77	38	56
Cl	202	198	123	216				43	18	27
NO_3								0.5	0.2	0.3
								4.2	0.1	2.2
TDS				1190				295	175	231
Hardness			85	57.5	1248	200	1550	188	104	143
Alkalinity					212	114	530			
pH	7.6	7.6	7.8	8.65				6.7	7.9	7.9
Temperature, F					71	73				
Turbidity, JTU				12	15.1	18.9	25			
Color				60						
Conductivity				1570	5670	426	7040			
TH (soap)	102	102								