2.4. HYDROLOGIC ENGINEERING

2.4.1. Hydrologic Description

2.4.1.1. Site and Facilities

The Fermi site is located adjacent to the western shore of Lake Erie (Figure 2.4-1). Prior to construction of Fermi 2, the site area was a lagoon separated from Lake Erie by a barrier beach, known as Lagoona Beach, which formed the eastern site boundary. The Fermi 2 preconstruction topography is shown in Figure 2.4-2. The lagoon was connected to Lake Erie by Swan Creek, a perennial stream that discharges into Lake Erie about 1 mile north of the Fermi plant site. The site for Fermi 2 was prepared by excavating soft soils and rock, and constructing rock fill to a nominal plant grade elevation of 583 ft. All elevations refer to New York Mean Tide, 1935. The topography of the developed site as of December 10, 1972, is shown in Figure 2.4-3.

Category I structures housing safety-related equipment consist of the reactor/auxiliary building and the residual heat removal (RHR) complex. These structures are indicated in Figure 2.1-5. The plant site is not susceptible to flooding caused by surface runoff because of the shoreline location and the distance of the site from major streams. Plant grade is raised approximately 11 ft above the surrounding area to further minimize the possibility of flooding. Flooding of the site is conceivable only as the result of an extremely severe storm with a storm-generated rise in the level of Lake Erie. Protection of safety-related structures and equipment against this type of flooding is provided through the location, arrangement, and design of the structures with respect to the shoreline and possible storm-generated waves.

After the excavation of topsoil, peat, and soft clay, construction of the plant site to grade Elevation 583 ft (nominal) was accomplished using the following fill materials:

a. Crushed rock (1-1/2-in. maximum) within 10 ft from the building walls (water has been observed to run off rather than drain through this evenly graded crushed rock)

b. Crushed rock (6-in. maximum) inside the perimeter road (surrounding the plant main structures), except adjacent to buildings (this permits water to drain quite well)

c. Quarry run rock for most fill areas outside the perimeter road (surrounding the plant main structures) (providing good drainage for water under almost all circumstances)

d. Topsoil for grass was placed on a layer of 1-ft-deep crushed-rock fill, 1-1/2-in. maximum, to avoid being washed down.

Roof water that is collected through drainage systems from all structures and catch basins inside the perimeter road is collected and routed to the station storm-water drain system to prevent ponding of water adjacent to structures. Water in the plant storm-water drain system is then discharged into the overflow canal. In grassy areas outside the perimeter road, and in gravel areas, catch basins discharge water into the quarry run fill. In paved areas, the catch basins are usually tied to the storm-water drain system. The plant circulating water is treated within the closed loop circulating water system, which includes the 5.5-acre circulating water reservoir.

2.4.1.2. Hydrosphere

2.4.1.2.1. Regional Conditions

The region of the Fermi site is located within the western part of the Lake Erie drainage basin. The divide between the Lake Michigan and the Lake Erie watersheds lies about 50 miles west of the site. Perennial streams in the region generally flow in a southeasterly direction and discharge into Lake Erie. Tributaries of these streams are intermittent and form a dendritic drainage pattern.

The average precipitation in the region ranges from 30 in. to 36 in./yr (Subsection 2.3.1.2). Average annual runoff ranges from 10 to 16 in. Infiltration is highest in the western part of the region in areas where permeable soils occur in end moraines and beach lacustrine deposits. High runoff coefficients are characteristic of the relatively impermeable lacustrine soils in the eastern part of the region.

2.4.1.2.2. Swan Creek

The Fermi site is in the Swan Creek drainage basin. The watershed is an area of 109 square miles, elongated in shape from northwest to southeast (Figure 2.4-4). The basin is about 25 miles long with a maximum topographic relief of about 130 ft. The drainage area topography is flat to gently undulating and varies from about 700 ft elevation in the upper watershed to about 570 ft elevation at Lake Erie.

Land in the basin is mixed in use for residential, commercial, industrial, and agricultural purposes. The surface soils are primarily lacustrine clay with some lacustrine sand ridges at the head of the watershed. The infiltration capacity of the basin soils is low. Surface drainage is poor and drainage ditch improvements are common in the upper part of the basin. Stream channel flow is retarded by typical vegetative cover of deciduous trees and brush undergrowth. There are no flow-control structures on Swan Creek. Stream level near the site is controlled by the level of Lake Erie.

Gages were placed along Swan Creek in 1971 and the collected data indicate that runoff is greatest during the spring and early summer (Reference 1). Data on the adjacent River Raisin and Huron River also indicate that runoff is highest during spring and summer. However, Swan Creek stream flow is normally too low for water supply use.

2.4.1.2.3. Lake Erie

2.4.1.2.3.1. Lake Characteristics

Lake Erie is approximately 240 miles long and has a mean width of 40 miles. The lake is divided into three principal subbasins: (1) a small, shallow basin at the west end which borders the site and is partially restricted by a chain of barrier beaches and islands; (2) a flat, unrestricted, and rather shallow basin in the center; and (3) a small, relatively deep eastern basin. The average depth of the lake is 61 ft and the maximum depth is 210 ft. The longitudinal axis of the lake trends northeast-southwest, a direction coincident with strong and persistent winds that predominate under normal meteorological conditions. Wind

stresses acting upon the lake surface over a sustained period can have a considerable effect on the level of the lake.

The most significant lake level variations are observed mainly at the western and eastern ends of the lake and are caused by transport of water as a result of sustained wind actions. Historical records show that in about 96 percent of all extreme cases, high water occurred at the eastern end of the lake and low water occurred at the western end. This is a result of the predominantly westerly winds causing the lake to set up at the eastern end.

The lake bottom in the vicinity of the site slopes very gently toward the east, reaching a depth of approximately 12 ft about 1/2 mile offshore. The soil deposits below the west end of the lake consist primarily of sand with intermittent layers of gravel and/or clay.

Two primary current patterns exist in the Lagoona Beach embayment. Winds moving from the northwest clockwise through northeast result in a general southwestward airflow over the entire embayment. This airflow creates the pattern of water movement shown in Figure 2.4-5. When the winds are from east-southeast clockwise through west, northward longshore currents are found to exist with a pronounced clockwise eddy formed south of the Point Mouillee marshes. This current pattern is shown in Figure 2.4-6.

When onshore winds from east clockwise through east-southeast and offshore winds from west-northwest clockwise through northwest occur, phase systems of current flow develop that produce variable patterns. The longshore currents shift from one primary current pattern to the other, reflecting changes in the local wind system. These phase changes are generally of short duration. Under ice cover, variations occur in the southward current flow and result in divergence of the currents immediately south of the existing plant intake and convergence north and east of Pointe Aux Peaux as shown in Figure 2.4-7.

2.4.1.2.3.2. Water Use

The use of potable and agricultural surface water within 10 miles of the plant site is presented in Subsection 2.1.4.2. Surface-water users withdrawing water from intakes in Lake Erie are the only surface-water users subject to the effects of accidental or normal releases of contaminants from the plant into the hydrosphere. The existing intakes along the western shore of Lake Erie have been examined to ensure that the dilution capacity of Lake Erie is sufficient to preclude adverse effects on users from releases of contaminants (Subsection 2.4.12). It is expected that future intakes will be located in the same approximate area and likewise will not be exposed to adverse effects of contaminants.

Municipalities with Lake Erie intakes, listed in Table 2.1-12, are located as shown in Figure 2.1-20. The municipal water intake nearest to the plant is the Monroe intake near Pointe Aux Peaux, approximately 2 miles southeast of the site, as shown in Figure 2.4-1. The Toledo intake is located about 18.6 miles due south of the plant site. The 1972 annual withdrawals at the Monroe and Toledo intakes were 2000 x 10^6 gal and 29,200 x 10^6 gal, respectively.

2.4.1.2.4. Ground Water

Regional ground water features are discussed in Subsection 2.4.13.1.1. Ground water in the site area occurs in a dolomite aquifer, underlying a mantle of relatively impermeable glacial deposits and recent sediments. This mantle ranges up to 40 ft in thickness. Water wells are

of low yield and the water is highly mineralized. The aquifer characteristics and ground water uses are described in more detail in Subsection 2.4.13.2.

2.4.2. Floods

2.4.2.1. Flood History

2.4.2.1.1. Maximum Mean Monthly Lake Levels

Based upon data collected by the U.S. Lake Survey, Detroit, Michigan (Reference 2), the highest observed monthly mean water level during the period of record from 1860 to 1973 was +4.9 ft above Low Water Datum. This level occurred during June 1973, at Monroe, Michigan. During 1973, the monthly mean water level varied between +3.0 and +4.9 ft above Low Water Datum, a vertical variation of 1.9 ft (Figure 2.4-9).

2.4.2.1.2. Maximum Wind Tide

Lake gaging records at Monroe have been collected for the periods from 1932 to 1939 and from 1952 to the present. Data from gages at Gibraltar and Toledo have been in existence since 1897 and have been correlated with records from the Monroe gage. Based on this relationship, the calculated maximum wind tide at Monroe was +4.5 ft on January 30, 1939. In an earlier report covering the period 1886 to 1896, a maximum wind tide of +5.5 ft was reported at Monroe. The description of the easterly gales that produced this wind tide suggests that they were more intense than those reported during the past 77 years. Therefore, it is reasonable to accept +5.5 ft (Elevation 576.0 ft) as the maximum wind tide occurrence since 1886.

2.4.2.1.3. Seiche History

Seiche history is discussed in Subsection 2.4.5.2.

2.4.2.1.4. Swan Creek

Complete flood data are not available for Swan Creek as gages were not installed until 1971. Long-term information exists from gages on adjacent drainage basins. On the River Raisin near Monroe, the largest flood (record begins in 1938) occurred on March 29, 1950, and the second largest on April 6, 1947. On the Huron River at Ann Arbor, the largest flood (record begins in 1918) occurred on April 5, 1947. Maximum annual floods occur principally in April and May. Discharge frequencies at the mouth of Swan Creek, estimated using standard methods (References 3 and 4), are shown in Table 2.4-1.

The estimated 100-year frequency discharge of 9300 cfs on Swan Creek is significantly less than the probable maximum flood (PMF) flow of 89,000 cfs (Subsection 2.4.3.4). In Subsection 2.4.3.5, it is demonstrated that the PMF flow on Swan Creek could not cause flooding at plant grade Elevation 583.0 ft. Therefore, water levels for the estimated discharges in Table 2.4-1 are not pertinent to site flood considerations.

2.4.2.1.5. Recent Storms

2.4.2.1.5.1. April 1966 Storm and Flood Analysis

On April 27, 1966, a persistent storm system moved into the Lake Erie drainage basin. During the month of the storm, the mean lake level at Toledo, Ohio, was 1.7 ft above the Low Water Datum of 570.5 ft. The maximum surge on Lake Erie occurred at Toledo while proportionately smaller surges were measured at distances from Toledo. The water level at Toledo reached 577.50 ft, which was 7.0 ft above the datum. The surge was driven by steady northeast winds with a directional duration of about 48 hr. At the time of peak surge, 1000 hr on the 27th, the maximum wind velocity measured at the Detroit River Light Station was 38 knots. However, a maximum wind velocity of 42 knots from the east-northeast was measured at 1300 hr, by which time the surge elevation had dropped to 575.93 ft.

Wave heights ranging from 6 to 7 ft were reported at the Toledo Harbor Light Station. To supplement the available wave data, a wave hindcast analysis was performed for the Fermi site. As discussed above, the times of peak surge and of peak wind velocity do not coincide, and this was considered in the hindcast analysis. The critical wind speed measured at the Detroit River Light Station was 38 knots from the northeast. This wind speed was increased by a factor of 1.30 to obtain a velocity representative of open-water conditions. The fetch aligned with the wind direction was 51,650 ft long and had associated with it a depth of approximately 13 ft at high water. A significant wave height and period of 3.8 ft and 3.2 sec, and a maximum wave height and period of 6.8 ft and 3.8 sec, would have been generated during this storm. Because the shoreline north of the Fermi site is oriented northeast, the waves that approached the site would have been attenuated by refraction and by the available depth of water over the sloping lake bottom. A conservative approximation of the lake bottom slope in this area is 1:100. Using this slope and the maximum wave period, the maximum supported wave height reaching the beach at the highest water level would have been about 1.3 ft. Waves larger than this would have broken too far seaward of the beach berm to have affected the site. The maximum runup elevation that would have been reached during this storm is 579.6 ft. This elevation is considerably less than the plant grade at the Fermi site of 583.0 ft and the probable maximum meteorological event (PMME) water level of 586.9 ft (Subsection 2.4.5).

2.4.2.1.5.2. November 1972 Storm and Flood Analysis

On November 13, 1972, a sudden storm moved into the Lake Erie drainage basin. The storm produced widespread flooding after the storm winds shifted from south to northeast, resulting in local evacuation within the low-lying areas along the western and southwestern shores. The total effect of the storm was that of a wind tide plus the abnormally high water level of Lake Erie, which existed at the time. In November, the mean lake level at Toledo was 3.6 ft above the Low Water Datum of 570.5 ft. The maximum surge on Lake Erie occurred at Toledo, while proportionately smaller surges were measured at distances from Toledo. The water level at Toledo reached 577.9 ft, which is 7.4 ft above the datum, while the maximum level at the Fermi site was 576.8 ft, which is 6.3 ft above the datum. Marblehead and Cleveland, Ohio, experienced maximum surges to Elevations 577.0 and 576.2 ft, respectively. The surge was driven by northeast winds with a directional duration of approximately 24 hr and a maximum velocity of about 40 knots over the central portion of the lake.

For most of November 12, 1972, winds were light and out of the southwest. Very late on the 12th and throughout the 13th, winds shifted gradually to northwest, then to northeast. By midday on November 13, the northeast winds were established and the velocity increased to 20 knots. The water level began rising at the Fermi site at 0800 hr on November 13. The maximum wind speed at Toledo was 25 knots and was reached early on November 14. By midday on the 14th, when the wind direction was changing to north, the water level at the Fermi site had reached its maximum elevation, 576.8 ft. The water level dropped rapidly, reaching a minimum level of elevation at 1800 hr on the 14th. Wind direction remained northerly throughout the 15th and velocity varied from 5 to 14 knots. Secondary and tertiary seiches were experienced on the 15th, but decayed rapidly from bottom friction. The troughs of these seiches resulted in lake elevations of 573.5 and 573.3 ft at the Fermi site. By November 16, the water level had stabilized at approximately Elevation 574.3 ft.

Waves during this storm were not measured at the site. Sufficient data describing the storm are available to hindcast the probable wave attack at the site. Waves were estimated at the Detroit River Light Station as ranging between 5 and 8 ft. Wind speed reached a maximum of 35 knots from the northeast at the Detroit River Light Station while Toledo Express Airport reported a maximum of 25 knots from direction N50°E. Applying a factor of 1.3 to the Detroit River Light Station yields an over-water wind velocity of 45.5 knots. The fetch aligned with the wind direction was approximately 51,000 ft long and had associated with it a depth of approximately 20 ft at high water. A significant wave height and period of 4.2 ft and 3.3 sec, and a maximum wave height and period of 7.6 ft and 4.0 sec, would have been generated during this storm.

The waves that approached the Fermi site would have been limited in height by the available depth of water over the gradually sloping lake bottom. Figure 2.4-10 shows the bathymetry offshore of the site.

A conservative approximation of the lake bottom slope in this area is 1:100. Using this slope and the maximum wave period, the maximum supported wave height reaching the beach at highest water level would have been 1.7 ft. Waves larger than this would have broken too far seaward of the beach berm to have affected the site.

The maximum runup elevation which would have been reached during this storm is 579.6 ft. This elevation is considerably less than the plant grade at the Fermi site of 583.0 ft and the PMME water level of 586.9 ft.

2.4.2.1.5.3. April 1973 Storm and Flood Analysis

Another storm moved into the Lake Erie Basin on April 9, 1973. Although this storm was less intense than the November 1972 storm, its total impact was nearly equal to the November storm because of the extremely high static lake level at the time.

In April 1973, the mean lake level at Toledo was measured by the U.S. Lake Survey as +4.76 ft above the Low Water Datum of 570.5 ft. The maximum surge associated with this spring storm was measured as +3.3 ft at Toledo, which brought the total stillwater level to 578.6 ft. This is 0.7 ft higher than the level reached by the November l972 storm.

On April 8, 1973, wind speeds ranged from 15 to 20 knots, blowing steadily from the northeast. On the morning of the 9th, the wind speed increased, reaching a maximum value

of 35 knots and shifting gradually to the east-northeast by 1430 hr. The water level began rising at Toledo, Ohio, at 0100 hr on April 9 and reached maximum Elevation 578.57 ft at 1600 hr on the 9th. The water level dropped rapidly, reaching minimum level Elevation 573.2 ft at 0100 hr on the l0th.

Secondary and tertiary seiches were experienced on the 10th, but decayed rapidly from bottom friction. By April 11, the water level had stabilized at approximate Elevation 574.6 ft. At the height of the storm, an 8-ft wave height was reported at the Detroit River Light Station.

To supplement the available wave data, a wave hindcast analysis was performed for the Fermi site. The maximum wind speed measured at the Detroit River Light Station was 35 knots from direction N67.5°E. This wind speed was increased by a factor of 1.30 to obtain an over-water velocity. The fetch aligned with the wind direction was 66,900 ft long and had associated with it a depth of approximately 20 ft at high water. A significant wave height and period of 4.8 ft and 3.6 sec, and a maximum wave height and period of 8.6 ft and 4.3 sec, would have been generated during this storm.

The waves that approached the Fermi site would have been limited in height by the available depth of water over the gradually sloping lake bottom. A conservative approximation of the slope of the lake bottom is 1:100. Using this slope and the maximum wave period, the maximum supported wave height reaching the beach at highest water level would have been 2.0 ft. Waves larger than this would have broken too far seaward of the beach berm to have affected the site. The maximum runup elevation that would have been reached during this storm is 581.7 ft. This elevation is less than the plant grade at the Fermi site of 583.0 ft and the PMME water level of 586.9 ft.

2.4.2.1.5.4. June 1973 Storm and Flood Analysis

High static lake levels continued through 1973. During June the mean lake level measured at Toledo by the U.S. Lake Survey was approximately 4.9 ft above the Low Water Datum of 570.5 ft. The earlier April 1973 storm occurred at a time when the lake was approximately 4.8 ft above the Low Water Datum. The maximum instantaneous surge associated with this June storm was measured at +3.4 ft at Toledo, which brought the total stillwater level to 578.7 ft. This was 0.1 ft above the April 1973 storm and 0.8 ft higher than the November 1972 storm.

At the Fermi site, maximum stillwater levels recorded by the U.S. Lake Survey reached a peak hourly reading of 577.75 (Low Water Datum) at 0200 hr on June 17, 1973. The Fermi water-level recorder does not record instantaneous water levels; however, interpolation from stations at Toledo, Ohio, and Gibraltar, Michigan, yields an instantaneous high of approximately 578.6 ft. Detroit area newspapers reported a maximum flood stage of 578.4 ft.

Wind speeds with an easterly component at the west end of Lake Erie between June 17 and June 18 were generally light to moderate. The Toledo Express Airport recorded fastest 1-minute velocities of only 9.6 knots, while the Detroit River Light Station recorded velocities between 10 and 15 knots. In addition, the Canadian government reported easterly gusts to 34 knots with an average of 20.9 knots at their Southeast Shoal lighthouse near Pt. Pelee, Ontario. The duration of these easterly winds was about 25 hr with peak velocities reached in the first 6 hr.

Winds at the east end of the lake, at Buffalo, were only slightly higher but maintained an easterly component for approximately 34 hr. It was this long-duration, moderate-wind regime at the east end of Lake Erie that was primarily responsible for the flooding at the west end. Buffalo reported east winds 12 hr before Toledo. The east winds from Buffalo were met by westerly winds from Toledo, which resulted in a temporary water buildup (to Elevation 576.3 ft 4 in.) at Cleveland. When the Toledo winds finally switched from west to east, the light to moderate velocities were enough to push the surge into the western end of the lake.

Wave heights, which were estimated during the storm at the Detroit River Light Station, ranged from 2 to 5 ft. To supplement available data, a wave hindcast analysis was performed at the Fermi site. Assuming a maximum steady-state wind velocity of 21 knots blowing from the east (N90°E), and applying a factor of 1.3, an over-water wind velocity of 27.3 knots is obtained. The maximum fetch aligned with the wind direction was 199,500 ft and had associated with it a depth of approximately 25 ft at high water. A significant wave height and period of 3.9 ft and 3.2 sec, and maximum wave height and period of 7.0 ft and 3.8 sec, would have been generated during this storm.

The waves that approached the Fermi site would have been limited in height by the available depth of water over the gradually sloping lake bottom. A conservative approximation of the slope of the lake bottom is 1:100. Using this slope and the maximum wave period, the maximum supported wave height reaching the beach at highest water level would have been 1.3 ft. Waves higher than this would have broken too far seaward of the beach berm to have affected the site. The maximum runup elevation that would have been reached during this storm is 581.0 ft. This elevation is less than the plant grade at the Fermi site of 583.0 ft and the PMME water level of 586.9 ft.

2.4.2.1.5.5. April 1974 Storm and Flood Analysis

In 1974 the highest water level measured by the U.S. Lake Survey at Toledo occurred on April 8 at 12 noon. The maximum reading was the result of sustained high static lake levels and an early spring storm.

In March and April the mean lake level at Toledo was approximately 4.4 ft above the Low Water Datum of 570.5. The maximum surge associated with the storm that moved through the area on April 7 and 8 was measured at +3.6 ft, which brought the total stillwater level to 578.5 ft. This was 0.2 ft below the June 1973 storm and 0.1 ft below the spring storm of April 1973.

At the Fermi site, the maximum stillwater level recorded by the U.S. Lake Survey was at Elevation 577.6 ft, which occurred at 12 noon on April 8.

Fastest 1-minute wind speeds measured at the Toledo Express Airport had a northeasterly direction and obtained a maximum of 26 knots with an average of 16.3 knots. At the Detroit River Light Station, a maximum wind velocity of 28 knots from the northeast and an estimated wave height of 4 to 5 ft were recorded at l030 hr on April 8. At 1630 hr on April 8, the light station recorded an east-northeast wind at 25 knots and a wave height of 5 to 6 ft. At this time water levels were already dropping at both Toledo and the Fermi site.

To supplement the available wave data, a wave hindcast analysis was performed for the Fermi site. Assuming a maximum steady-state wind velocity of 28 knots from direction

N67.5°E and applying a factor of 1.3, an over-water wind velocity of 36.4 knots is obtained. The maximum fetch aligned with the wind direction was 66,900 ft long and had associated with it a depth of approximately 20 ft at high water. A significant wave height and period of 3.8 ft and 3.2 sec, and a maximum wave height and period of 6.8 ft and 3.7 sec, would have been generated during this storm.

The waves that approached the Fermi site would have been limited in height by the available depth of water over the gradually sloping lake bottom. A conservative approximation of the slope of the lake bottom is 1:100. Using this slope and the maximum wave period, the maximum supported wave height would have been 1.6 ft. Waves larger than this would have broken too far seaward of the beach berm to have affected the site. The maximum runup elevation that would have been reached during this storm is 581.3 ft. This elevation is less than the plant grade at the Fermi site of 583.0 ft and the PMME water level of 586.9 ft.

2.4.2.2. Flood Design Consideration

2.4.2.2.1. Conditions Considered

The following basic types of hypothetical flooding conditions were considered in the design:

a. The PMF of 89,000 cfs on Swan Creek coincides with the mean monthly maximum water level of 575.3 ft in Lake Erie. In the discussion of backwater computations (Subsection 2.4.3.5), the resulting PMF flow elevation of 577.3 ft would provide a safety margin of 5.7 ft. Even by the use of a conservative slope/area computation (Subsection 2.4.3.5), the PMF elevation would be less than 582 ft, or 1 ft below plant grade at 583 ft and 1.5 ft below the elevation of plant door sills

b. The maximum probable wind tide of 11.6 ft coincides with a maximum monthly mean lake level of 575.3 ft. The resulting stillwater flood elevation at the plant site area in this case is 586.9 ft, or 3.90 ft above the plant grade elevation (Subsection 2.4.5.3)

c. Local probable maximum precipitation (PMP) runoff on the plant site coincident with runoff from the 2-square mile area above the plant site, assuming blockage of plant drainage, would result in no adverse effects on the safety-related (Category I) facilities. The estimated PMF of 25,300 cfs with a corresponding elevation of less than 582 ft, and the 15-minute PMP of 4.9 in. over the plant site with a grade elevation of 583 ft and door sills at 583.5 ft would not result in adverse plant site flooding, as further discussed in Subsection 2.4.2.3. The temporary local water buildup due to the failure of the plant drainage system will flow into the lower land and swamps at the northern end of the plant area and eventually discharge into Lake Erie through estuaries. The local temporary water buildup elevation will be substantially lower than the flood elevation due to the maximum wind tide, as described in item b. above

d. The potential dam failure effect is not applicable, as described in Subsection 2.4.4

e. The water level at the site is controlled by Lake Erie. The PMF flow from Swan Creek has no significant effect on the design water level at the site. The maximum lake stillwater level due to storm surge is Elevation 586.9 ft (Subsection 2.4.5.3). Plant grade is at Elevation 583.0 ft. At plant grade elevation, the lake water would extend approximately 2.5 miles inland from the plant site (Figure 2.4-11) and even further inland at maximum stillwater level.

The case (item b) above is clearly the most critical condition and is defined as the PMME.

2.4.2.2.2. Reactor/Auxiliary Building Flood Criteria

The Category I reactor/auxiliary building, which houses safety-related systems and components, is designed against flooding to Elevation 588.0 ft, or 1.1 ft above the PMME stillwater flood elevation of 586.9 ft. All doors and penetrations through the outside walls below the design flood elevation are of watertight design. All safety-related systems and equipment located inside this Category I structure are protected from the PMME flood. The reactor/auxiliary building is also designed to withstand wave action associated with this flooding. Maximum wave effects and forces are discussed in Subsection 2.4.5.4.

All interior floor drain systems inside the reactor/auxiliary building are not connected to the yard storm drainage system and, therefore, no potential water backflow into the structure is anticipated during the design flood condition. Shore protection is not required to preclude flooding of this structure.

The reactor/auxiliary building has only a few essential penetrations in the exterior walls. All of these penetrations below Elevation 588 ft are watertight.

The presence of the turbine building prevents waves and wave runup above the sill elevations on the east wall of the reactor/ auxiliary building, thereby preventing flooding of the buildings. The south wall of the reactor/auxiliary building has two large openings, two rail pockets with waterproofed seals and several waterproofed pipe-sleeved openings. These large openings are in an air-locked rail-car door and an air-locked personnel door. Both of these doors, however, will be air-locked and completely waterproofed to preclude wave runup flooding.

The reactor/auxiliary building roof is designed for a live load of 30 lb/ft^2. This load is equivalent to approximately 6 in. of water, or its equivalent in snow, or snow and ice load combined. Roof drains are designed for a rainfall of 4 in./hr. The reactor building roof water drains through openings in the parapet wall into scuppers and then down through conductors to the auxiliary building roof. Roof drains in the auxiliary building roof carry the runoff into the buried site drainage system by first passing through the turbine building roof drainage system.

2.4.2.2.3. Residual Heat Removal Complex Flood Criteria

The RHR complex is watertight to Elevation 590.0 ft. The north, south, and west walls have no openings. The east wall has approximately 30 waterproofed pipe-sleeved openings. The east wall also has four sets of double 3 ft by 7 ft doors for access to the building. These doors are normally closed and locked, and have their thresholds at Elevation 590.0 ft and extend to Elevation 597.0 ft. They are of steel construction and are shielded behind

reinforced-concrete missile walls. The east wall also has eight 4" diameter openings with water tight seals located within each of the two RHR cable vaults at elevations above 590'-6".

Waves reaching the east wall of the RHR complex across the flooded site would be diminished considerably by the stairs, the missile wall, and the landing at Elevation 590.0 ft in front of the doors. The insignificant amount of runup above the flooded elevation of 586.9 ft, or generated by the reduced waves, may find its way through the door threshold and door jambs, at Elevation 590.0 ft, and be diverted into the floor drain system in the building. The structure is also designed to withstand the wave action associated with this flooding. Shore protection is not required to preclude flooding of this structure.

The roofs of the RHR complex are provided with an adequate number of drainage pipes to pass runoff resulting from the PMP. The PMP was obtained from U.S. Weather Bureau (National Oceanic and Atmospheric Administration) information (Reference 5). Further, the storm-drainage provisions surrounding the RHR complex are designed to pass the discharge from the drain pipes as well as the runoff from surrounding areas. The plant area drainage system is designed so that there is no possibility of ponding near the RHR complex. The roofs of the RHR complex are designed for a postulated maximum ice and snow load of 70 lb/ft^2. This load is based on the simultaneous accumulation of the most severe postulated ice resulting from the mechanical draft cooling towers drift loss (21 lb/ft^2) plus the seasonal snowpack (30 lb/ ft^2), and on an additional ice load (19 lb/ft^2).

The mechanical draft cooling tower drift loss is based on an assumed drift loss of 0.015 percent, with the fans operating at full speed. For evaluating the ice loading on the RHR complex roof, a conservative value of 0.1 percent for drift loss was used at full speed. Under freezing conditions, the fans operate at half speed or are completely shut off. The total water loss under these conditions is less than 390 gal/hr. Based on the above, it is estimated that, with two towers operating for 30 days with no wind drift, and with the temperature below freezing, the maximum ice accumulation is less than 4-1/2 in. This amount of ice is equivalent to about 21 lb/ft^2 live load.

The seasonal snowpack load is based on results of reported research (Reference 6). According to this reference, the seasonal snowpack load is 30 lb/ft^2.

2.4.2.2.4. Category I Yard Structures Flood Design Criteria

The Category I piping and electrical ducts between the RHR complex and the reactor building are below the site flood elevation of 586.9 ft during the PMME. The RHR supply, RHR return, and emergency equipment service water pipelines to both divisions will continue to function during the flood.

There are two sets of Category I ductbanks between the RHR complex and the Reactor/Auxiliary building, with a Division I and Division II ductbank in each set. In each case, the buried cable ducts between the RHR complex and the Reactor/Auxiliary building provide adequate cable separation to maintain independence of redundant circuits.

The first set of ductbanks was installed during plant construction. The physical separation of the two redundant, below-grade circuits is 30 ft at the point the cable ducts leave the southeast corner of the reactor building. The ducts make a sweeping bend with a minimum

separation of 20 ft between them. After the bend, the ducts parallel the reactor building in a westerly direction, with 24-ft separation. This separation is constant until the ducts pass under the rail-car air lock, where the separation widens until the ducts enter (still below grade) the RHR complex.

Each circuit is separately housed in a cast-in-place, rectangular reinforced-concrete duct. The duct is covered by successive layers of compacted rock fill placed up to the finished site grade of 583.0 ft. The duct runs vary in elevation from 573.0 ft minimum to 580.0 ft maximum. Since maximum ground water elevation is 576.0 ft, the cables are not specifically designed for continuous underwater service. For low voltage power, control and instrumentation cables, there is no long term mechanism for water related insulation degradation due to lack of voltage stressor or a credible common mode failure mechanism. Therefore, low voltage cables perform their design functions while their external surface remains continuously wetted due to surrounding water. 4160-V essential power circuits are not routed within these ductbanks.

The second set of ductbanks, associated manholes, and cable vaults is installed above the maximum ground water elevation of 576.0 ft with ducts sloped to the manholes, such that circuits contained are not subject to continuous wetting. These are also cast-in-place, rectangular reinforced concrete ductbanks, but are located with the ductbank top approximately six inches below the surface and manhole covers at grade level. The ductbanks rise above grade and enter above ground cable vaults at the RHR complex and also rise above grade at the entrance to the Reactor/Auxiliary building cable vaults. 4160-V essential power circuits are routed within these ductbanks.

The minimum elevation for cable termination in either the RHR complex or reactor building is 588.7 ft, which is above the site maximum probable stillwater elevation of 586.9 ft.

2.4.2.2.5. Site Drainage Flood Design Criteria

The storm drainage system is not used to protect Category I structures from local PMP flooding, as further discussed in Subsection 2.4.2.3. Inlet manholes in the immediate plant vicinity are located at the low points of relatively flat roadside and railroad track areas, and in local area depressions. The storm-drainage conduit discharges westward into the existing overflow canal for Fermi 1 and eventually into Lake Erie through estuaries. The storm-drainage system is designed as a gravity system with a minimum velocity of 3 fps flowing full for a rainfall intensity of 4 in./hr. Runoff coefficients used are 1.0 for roofs and paved areas and 0.5 for gravel and grassed areas. The closed storm-drainage system provides the normal means of drainage for the plant site and building roofs.

The sedimentation potential of the site drainage system for anticipated rainfall conditions is negligible since the site consists principally of firmly compacted crushed-rock fill and grassed areas, and the slopes of the ditches feeding the inlet of manholes are relatively flat. The resulting velocity of the drainage flow is nonscouring. Riprap or paving is provided for protection of outlet ends at all discharge points of the storm sewer system.

2.4.2.3. Effects of Local Intense Precipitation

Flooding due to a local PMP on the adjacent 2-square mile drainage area west of the plant site, as shown in Figure 2.4-4, was examined. The local PMP shown in Table 2.4-2 was determined by use of Reference 5. The hourly distribution of the maximum 6-hr rainfall was determined by procedures presented in Reference 7. The shorter 15-minute-duration PMP was extrapolated by use of similar procedures. Due to its small area, the rational formula with a runoff coefficient of 1.0 and concentration time of 15 minutes was applied to compute the peak discharge (Reference 8). The maximum PMP intensity of 15 minutes is assumed to be 4.9 in., as shown in Table 2.4-2. The calculated peak discharge due to the local PMP is 25,000 cfs, which is 10,000 cfs greater than indicated by the PMF peak envelope curve for the Great Lakes region. The Great Lakes PMF peak discharge envelope curve indicates a maximum flow of 15,000 cfs, which represents a more severe flood than would result from the relatively flat 2-square mile local area if determined by the unit hydrograph PMP calculation procedure.

The calculated peak discharge due to the local PMP is 25,000 cfs. Assuming, conservatively, that the peak discharge would pass the plant site only along the axis of the overflow canal (Figure 2.1-5), a hypothetical cross section approximately 1 mile in length and normal to the axis of the overflow canal was constructed to intersect the southernmost chimney on the plant site and the intersection of Langton and Leroux roads to the west of the site (Figure 2.4-3).

Using the slope/area method and conservative values of slope and roughness coefficient, 0.001 ft/ft and 0.07, respectively, a flow of 31,500 cfs was determined as passing through the cross section with a maximum water surface elevation of 582 ft (New York Mean Tide, 1935). The peak flow due to a local PMP, 25,000 cfs, would pass through the cross section at an even lower water surface elevation. In this analysis, channel or cross-section bottom was assumed to be at maximum monthly mean lake level. And, as stated earlier, all flow due to a local PMP was assumed to pass through the hypothetical cross section. Under actual conditions, a peak flow due to the local PMP would flow both south of the plant site and to Lake Erie, as well as through the hypothetical cross section. Water surface elevations due to a local PMP would therefore be lower in actuality than those determined in our analysis.

At a hypothetical water surface elevation of less than 582 ft (New York Mean Tide, 1935), as determined in the above analysis, the maximum water elevation at peak flow due to a local PMP would be more than 1 ft below plant grade (583 ft, New York Mean Tide, 1935) and would not pose a threat to safety-related structures onsite.

With respect to that portion of a local PMP falling on the plant site itself, including roof structures, runoff overflowing the roof parapets and from the downspouts, assuming that the site drainage system was completely blocked, would flow overland under conditions of site gradient (Figure 2.1-5) to lower elevations surrounding the site and then to Lake Erie itself.

All door sills on safety-related structures are at least 6 in. above plant grade. Because there are no downspouts or scuppers located near doors on safety-related structures, ponded water under local PMP conditions, with the event of a blocked site drainage system, should drain overland, as described above, prior to reaching the base of door sills on safety-related structures.

The local PMP is shown in Table 2.4-2, and the description of the runoff model is given in Subsection 2.4.3.3.

The drainage system in the plant site area is designed with inlet manholes located at the low points of relatively flat roadside and railroad ditches and in local area depressions. The storm-drainage system is not used to protect Category I structures from local PMP flooding, as described in Subsection 2.4.2.2.

2.4.3. Probable Maximum Flood on Swan Creek

The PMF is an estimated flood that may be expected from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible in the region (References 5 and 7). The PMF on Swan Creek was estimated as the maximum flood runoff resulting from a PMP occurring on the entire drainage basin of 109 square miles, as shown in Figure 2.4-4.

2.4.3.1. Probable Maximum Precipitation

The estimation of a PMP includes both time and areal distributions. Due to its small drainage area (109 square miles), the PMP is assumed uniformly distributed throughout the entire Swan Creek watershed. The time distribution of a PMP is obtained as follows. The PMP for various durations shown in Table 2.4-3 was obtained from the all-season PMP (Reference 5). Its 2-hr time distribution for the maximum 6-hr rainfall and time sequence were based on procedures presented in Reference 7. Table 2.4-3 shows the synthesized PMP for the Swan Creek watershed.

2.4.3.2. Precipitation Losses

An estimate of precipitation losses was obtained using data from References 9 and 10 and studies of other similar areas. Surface soils in the Swan Creek drainage area are largely comprised of lacustrine clays, which have low infiltration capacity (Reference 11). The land use is estimated as follows: 30 percent small grain, 30 percent forage and pasture, 25 percent row crops, and 15 percent wooded land and buildings. Considering the Swan Creek type ground cover and soil surface as compared to similar type areas in other locations where studies have been made, minimum loss rates are higher in the summer months than in the winter months. These minimum losses can be characterized as follows.

a. Winter initial losses vary from 0.0 to 0.2 in., and winter infiltration losses vary from 0.01 to 0.02 in./ hr

b. Summer initial losses vary from 0.5 to 1.2 in., and minimum summer infiltration rates are approximately 0.05 in./hr.

The Swan Creek losses adopted are initial losses of 0.5 in. and an infiltration rate of 0.02 in./hr during the probable maximum storm. This is assumed as occurring during a wet period with the most favorable antecedent conditions when the moisture capacity of the topsoil would be essentially satisfied. The adopted minimum losses for the Swan Creek area assuming the most favorable (to high runoff) antecedent (ground and rainfall) conditions are based on a conservative estimate for these conditions. The Swan Creek rainfall-excess relationships were determined by use of the minimum conservative losses during the PMP

storm as shown in Table 2.4-4. The estimated precipitation losses and runoff are shown in Table 2.4-4.

2.4.3.3. Runoff Model

Because Swan Creek was ungaged prior to 1971, a synthetic unit hydrograph was developed for the 109-square mile basin, as shown in Figure 2.4-4, by using Snyder's method (Reference 12). The runoff was determined at the mouth of Swan Creek north of the site.

Figure 2.4-12 shows the synthetically derived unit hydrograph of 2-hr duration for the Swan Creek watershed. The hydrograph ordinates are shown in Table 2.4-4. Coefficients used in the derivation of the synthetic unit hydrograph are as follows: L = 25.4 miles, L_{ca} = 16.7 miles, C_t = 2.0, W_{50} = 16 hr, and W_{75} = 9 hr. The terms L and L_{ca} are distances measured on the U.S. Geological Survey (USGS) 7.5-minute topographical map for the site area. Time in hours, from start of rise to peak rate, or t_p, was determined using the formula

$$t_p = c_t(L * L_{ca})^{0.3}$$

The value of t_p was determined to be 12.3 hr using a basin parameter C_t of 2.0. Comparison of synthetic unit hydrograph values for Swan Creek with values for nearby stations with similar runoff characteristics as obtained from U.S. Army Corps of Engineers unpublished unit hydrographs is given in Table 2.4-5.

Table 2.4-5 illustrates the conservatism of the coefficients selected for the Swan Creek watershed. For example, a curve enveloping the q_p values would yield a unit hydrograph peak of about 3100 cfs for the 109 square miles as compared to the 4000 cfs peak adopted. The utilization of the extreme coefficient value was intended to include the possible nonlinear runoff response of Swan Creek due to high rainfall intensities.

2.4.3.4. Probable Maximum Flood Flow

The PMF for the 109-square mile watershed of Swan Creek was determined by appropriate application of the preceding analysis described in Subsections 2.4.3.1, 2.4.3.2, and 2.4.3.3. Base flow was assumed to be 100 cfs. The computed PMF hydrograph components are shown in Table 2.4-4.

The calculated basin-wide peak flow in Swan Creek due to the synthesized PMP is 89,000 cfs at the mouth of Swan Creek, as shown in Figure 2.4-13.

There are no dams or other regulating hydraulic structures on Swan Creek that could affect the hydrograph. The exact PMF stream course response cannot be assessed since Swan Creek has not been gaged for a sufficient period of time.

2.4.3.5. Water-Level Determinations

The water level at the site is controlled by Lake Erie. The PMF flow from Swan Creek has no significant effect on the design water level at the site. The maximum lake stillwater level due to storm surge is Elevation 586.9 ft (Subsection 2.4.2.2.1). Plant grade is at Elevation 583.0 ft. At plant grade elevation, the lake water would extend approximately 2.5 miles inland from the plant site (Figure 2.4-11) and even further inland at maximum stillwater level.

To estimate the maximum floodwater level, a section through the east end of the plant site and normal to Swan Creek was selected to compute backwater effects due to the PMF flow on Swan Creek. This section is 3.5 miles wide and is bounded by Port Sunlight Road to the north and Pointe Aux Peaux Road to the south (Figure 2.4-1). Neither of the roads was constructed as a flood-protection levee. In the vicinity of the control section, the land is flat, approximately at Elevation 572.5 ft (Figure 2.4-11).

The backwater calculations were done with the assumptions that the selected section has a water level at Elevation 575.3 ft, mean monthly maximum lake level, and the main plant structures are located 1500 ft west of this section. By applying the Manning formula (Reference 13) on a rectangular channel with a width of 3.5 miles and a bottom elevation of 572.5 ft, with a Manning's roughness coefficient of 0.07, the estimated rise of water level during a peak flood flow of 89,000 cfs is less than 2.0 ft. Therefore, the maximum flood level at the plant site due to the PMF flow from Swan Creek at the mean monthly maximum lake level is at approximately Elevation 577.3 ft, which provides a safety margin of more than 5.7 ft below the established plant grade of Elevation 583.0 ft.

The same procedures were applied using a higher peak flood flow of 115,000 cfs, resulting in an estimated maximum flood level at the plant site at Elevation 579.1 ft, which is 3.9 ft below the plant grade. Therefore, the PMF flow from Swan Creek has no flooding potential with respect to the plant site.

Additional computations, utilizing the slope/area method at a hypothetical cross section through Swan Creek above the plant site (Figure 2.4-4) determined that a flow of 106,000 cfs in Swan Creek would represent a maximum water surface elevation at the cross section of 582 ft (New York Mean Tide, 1935). The PMF of 89,000 cfs on Swan Creek (Subsection 2.4.3.4) should not cause flooding affecting safety-related structures at plant grade Elevation 583 ft (New York Mean Tide, 1935).

In the above computations by the slope/area method, a hypothetical cross section normal to Swan Creek and approximately 1.8 miles in length was chosen. Channel base or the bottom of the cross section was assumed to be at the elevation of the maximum monthly mean lake level. A slope of 0.001 ft/ft and a roughness coefficient of 0.07 were used in the computations.

2.4.3.6. Coincident Wind Wave Activity

A flood on Swan Creek would result in a landward extension of the lake. Therefore, wind activity determined for the lake would apply to the stream flood condition. Wave activity in Lake Erie is described in Subsection 2.4.5.4.

2.4.4. Potential Dam Failures (Seismically Induced)

There are no regulatory structures on Swan Creek. Nor are there dams on other streams or rivers in southeastern Michigan that should failure result because of seismic or other disturbances would affect water levels in Lake Erie along the plant shoreline.

2.4.5. Probable Maximum Surge and Seiche Flooding

2.4.5.1. Probable Maximum Winds and Associated Meteorological Parameters

Extensive studies have been made regarding the effects of wind setup on Lake Erie. Data developed by Platzman (Reference 14), which relate lake levels at Toledo and Buffalo to various wind conditions, were used to establish the wind setup for the site.

The Platzman one-dimensional wind setup model has been verified using four storms producing peak setup at Toledo (Reference 15). The model, valid for setup along the longitudinal axis of Lake Erie, has been shown to consistently calculate peak longitudinal setup greater than the measured peak longitudinal setup at Toledo when using the wind stress and bottom friction coefficients proposed by Platzman. Verification of this model is valid for input winds measured at the Ashtabula Coast Guard Station. The verification for one storm, and possibly a second, indicates that cross-lake wind setup can, at times, be significant and should be considered.

The conservatism of the model in predicting the longitudinal setup increases with increasing wind speed. For a maximum 3-hr average wind speed of 74 knots, the model is estimated to compute a longitudinal wind setup at Toledo 2 ft above the value which would be measured. Whereas an allowance should be made for the possibility of cross-lake setup occurring simultaneously with longitudinal setup at Toledo, an allowance is not required at the Fermi site near Monroe since Monroe is in the vicinity of the nodal point for cross-lake setup. The nodal point is the location where the change in stillwater level due to cross-lake setup is zero.

To establish meteorological conditions appropriate for calculation of the maximum probable wind setup for the site, winds with an easterly or northeasterly component that would be sustained for 6 to 9 hr were examined. The National Weather Records Center in Asheville, North Carolina, was commissioned to examine 25 years of wind records for eight stations in the vicinity of Lake Erie. The eight stations were Toledo, Windsor (Ontario), Sandusky, Cleveland, London (Ontario), Youngstown, Erie, and Buffalo. The National Weather Records Center tabulated (Reference 16) the speed, direction, and date of the fastest 1-minute wind having an easterly component.

The maximum, easterly 1-minute wind speeds observed for the 25-year period at the eastern four stations (London, Youngstown, Erie, and Buffalo) were 65, 37, 60, and 44 mph, respectively. The companion maximum, easterly 1-minute wind speeds observed at the western four stations (Toledo, Windsor, Sandusky, and Cleveland) were 40, 45, 35, and 35 mph respectively. Comprehensive analysis of these and other data (Reference 17) led to the conclusions that:

a. Maximum easterly wind speeds are substantially less than maximum westerly wind speeds

b. Maximum easterly wind speeds over the western portion of Lake Erie are somewhat less than maximum easterly wind speeds over the eastern portion of Lake Erie.

On this basis, a maximum, 1-minute easterly wind speed of 45 mph was selected as representative for the 25-year period of record for the site. This 1-minute value was converted to the probable maximum easterly wind as follows:

a. Overland wind speed was converted to over-water wind speed by multiplying the land value by 1.33. The maximum easterly wind speed over water is thus calculated as 60 mph. This wind speed is assumed to have a probability of once in 25 years

b. The maximum 1-minute easterly wind speed with a probability of once in 1000 years was calculated, using the method of Thom (Reference 18), to be 86 mph

c. A maximum 10-minute wind speed of 74 mph was calculated (Reference 19) by multiplying the maximum 1-minute easterly wind speed by 0.86

d. The 1000-year maximum easterly wind was taken as the maximum 10-minute wind speed of 74 mph.

The PMME data used to calculate the probable maximum wind tide at the Fermi site were obtained from the table of probable maximum wind estimates (over-water wind speeds) supplied by the AEC. The PMME wind speeds over the lake varied with time and distance along the lake axis. The peak 10-minute wind speed was 100 mph. Since the model used to calculate the probable maximum wind tide (Reference 14) is one dimensional, the PMME winds were directed along the axis of Lake Erie (N67.5°E). The PMME had a translational velocity of 20 mph moving from east to west, and duration of 60 hr.

2.4.5.2. Surge and Seiche History

2.4.5.2.1. Maximum Monthly Mean Lake Level

Historical maximum monthly mean water levels are discussed in Subsection 2.4.2.1.1.

2.4.5.2.2. Maximum Wind Tide

Historical maximum wind tides are discussed in Subsection 2.4.2.1.2.

2.4.5.2.3. Seiches

Seiches are periodic oscillations of the lake water level that are caused by changes in wind stress or barometric pressure acting upon the water surface. As the wind stress diminishes, the adverse gradient of the surface water cannot be maintained and an inertial surge of water occurs. Seiches also may result from very rapid changes in barometric pressure, usually associated with squall lines. However, sudden barometric disturbances are very infrequent on Lake Erie.

Analysis of gage records of Lake Erie indicates that the average period of oscillation for a seiche traveling between Toledo, Ohio, and Buffalo, New York, is approximately 14 to 15 hr. As a result of the greater depth of water at the east end of the lake and the generally higher wind speeds associated with the prevailing westerly winds, the maximum amplitudes of a seiche on Lake Erie occur at Buffalo.

Gages at Buffalo and Toledo indicate that the amplitude of the oscillations of a seiche decays rapidly with each subsequent oscillation. The rise in water level induced by the initial wind setup is greater than any subsequent rise associated with the seiche.

In addition to the general seiche that occurs over the entire lake surface, a local seiche may occur between the west end of Lake Erie and Point Pelee. Local seiches with amplitudes of up to 0.8 ft have been detected from gage records at Toledo and Monroe (Reference 20). These seiches can occur when the water body is in a state of equilibrium or constant stillwater level.

The stillwater level of Lake Erie near the Fermi site constantly changes in elevation, with respect to the rest of the lake during the PMME. This difference in water levels effectively damps out any seiche activity near the site. It is unlikely, therefore, that any seiche will occur simultaneously with the PMME. Consequently, for design purposes, no rise in water elevation from a seiche is considered.

2.4.5.3. Surge and Seiche Sources

The maximum PMME wind tide of 11.4 ft was calculated for the Fermi site with the PMME wind speeds as input to the verified Platzman one-dimensional wind setup model of Lake Erie (Reference 15). As an additional conservatism, the previously accepted wind tide of 11.6 ft was used for design purposes. This value does not include an allowance for cross-lake setup as none is required. Monroe is in the vicinity of the nodal point for cross-lake setup, where the change in stillwater level due to cross-lake setup is zero.

A total stillwater elevation of +16.4 ft (586.9 ft) was selected as the design maximum. This was based on the PMME defined by the AEC with a storm path along the axis of Lake Erie (N67.5°E). Elevation +16.4 ft results from a calculated wind tide of +11.6 ft superimposed on a maximum monthly mean lake level of +4.8 ft. This storm surge would occur at the Fermi site approximately 9 hr after the maximum wind reaches the shore. The storm surge hydrograph resulting from the PMME is shown in Figure 2.4-14.

No rise in water elevation resulting from a seiche was used in the design (Subsection 2.4.5.2.3).

2.4.5.4. Wave Action

2.4.5.4.1. Wind-Generated Waves

Wave characteristics are dependent upon wind speed, wind duration, water depth, and fetch length. Generated waves were calculated coincidental with the maximum storm surge hydrograph to determine the maximum flood elevations at the site. Fetch lengths were measured to the site from the axis of the lake (N67.5°E), from N78.75°E, and from due east (Figure 2.4-15). These fetches, hereafter referred to as degrees clockwise from north, have fetch lengths ranging from 11 to 33 nautical miles. Average lake depths range from 32 to 42 ft during probable maximum stillwater levels.

Using the AEC definition of probable maximum winds, component wind velocity profiles were plotted for fetch directions 67.5°, 78.75°, and 90.0° (Figure 2.4-16). Component wind velocities for fetch directions 78.75° and 90.0° were based on the wind velocity profile from 67.5°, the path of the storm.

The shallow water depths over the fetch approaching the Fermi site preclude deep-water wave activity; only shallow-water waves are generated during the PMME. The shallow-

water wave generation curves of Bretschneider (Reference 21) were used to calculate significant wave heights and periods (Figure 2.4-14). The generated wave height and period profiles have a phase shift in time of +1.5 hr over the wind profiles to allow for the generation and travel of waves to the site.

The significant wave height is the normal available parameter from statistical analysis of synoptic weather charts. Approximate relations of the significant wave heights to other parameters of the normal wave spectra in nature have been defined. Assuming that the most probable maximum wave height, H_m, is given by the deep water simplified theoretical solution of Equation 2.4-1, then the ratio of H_m to H_s is 1.8 to 1.

$$H_m = 0.707 H_s \sqrt{\log_e N} \qquad (2.4\text{-}1)$$

where

N = number of waves during a period of steady-state conditions

H_s = significant wave height

This value is conservative, as the wave spectrum curve is flatter for shallow-water conditions near the Fermi site than for deep-water conditions applicable to the solution. Curves of H_m are presented in Figure 2.4-16.

2.4.5.4.2. Design Waves

2.4.5.4.2.1. Selection Bases

Selection of design waves depends on the wave climate at the site, the structures being considered, and the available water depths fronting the structures. Generated wave conditions during the PMME occurrence, offshore of the site location (Figure 2.4-16), are propagated shoreward to the various plant structures. In selecting design waves for various structures, the possible range of wave periods, heights, and approach directions during various times of the storm are considered to occur at critical conditions.

2.4.5.4.2.2. Incident Wave at Shoreline

The maximum stillwater level and the maximum offshore generated wave height do not occur simultaneously. Therefore, various stillwater levels are considered in selecting the critical wave conditions. The maximum generated wave height, significant wave height, and wave period (offshore of the plant site) are 21.9 ft, 12.2 ft, and 9.0 sec, respectively. These occur during the stillwater level of 582.8 ft, 1.50 hr after the maximum winds have crossed the shoreline (Figure 2.4-14). During the maximum stillwater level of 586.9 ft and 9 hr after the maximum winds have crossed the shoreline, the maximum wave height, significant wave height, and wave period are 14.0 ft, 7.8 ft, and 7.7 sec, respectively.

Design waves were generated offshore of the site location from approach directions 67.5° (path of PMME), 78.75°, and 90.0°. There should be no significant wave action south of 110° (i.e., normal to the shoreline) during the occurrence of the PMME, as this direction is a 42.5° departure from the wind direction. Waves north of 67.5° also are insignificant because of diminishing fetch length, shallow water depths, and change of direction through wave refraction. An 8-sec wave period generated from 67.5° would approach the plant site

shoreline from due east because of refraction effects (Figure 2.4-10). A shorter wave period would not be affected by refraction as much as the 8-sec wave period.

As waves approach the shoreline, they start breaking in water depths approximately equal to their wave heights. Figure 2.4-14 shows breaking wave heights for shoreline toe elevations of 569 ft, 572 ft, and 575 ft. The upper breaking wave height limit considers the effects of wave setup. With continuous heavy wave action breaking against the shoreline, it is possible that the return flow of water lakeward will be slower, thus causing a pileup of water (wave setup) along the shoreline. The possibility of this wave setup was assumed to raise the stillwater level by an amount equal to one-tenth the breaking wave height. With this increase in stillwater level, a slightly higher wave could be supported before breaking.

In selecting the proper design wave that can attack the shoreline, Figure 2.4-14 is used. Design H_s and H_m curves were plotted from the maximum values of Figure 2.4-16. For a particular shoreline or shore barrier toe elevation, the breaking wave height is the controlling factor if it is less than the unbroken wave height during a given stillwater level. In Figure 2.4-14, which includes the storm surge hydrograph, the stillwater level is read off the right-hand ordinate while the wave parameters, H_m, H_s, and H_b, are read off the left-hand ordinate. In using either the significant wave height curve (H_s) or the maximum wave height curve (H_m), the breaking wave height curve (H_b) controls until it intersects (progressing positively from left to right along the TIME axis) the H_m or H_s curve. Thereafter, the unbroken wave height controls.

When using significant wave conditions and a toe elevation of 575.0 ft, the following applies:

a. For a time of +3 hr after the maximum winds reach shore, the design wave is a breaking wave of 7.9 ft to 8.6 ft, with a period of 8.8 sec, during a stillwater elevation of 584.0 ft

b. For a time of +9 hr, the design wave is a significant wave of 7.8 ft

c. The maximum design wave is a wave of 10.2 ft with a period of 8.4 sec and occurs during a stillwater elevation of 585.6 ft at a time of +5.1 hr.

2.4.5.4.2.3. Transmitted Wave

During the occurrence of the PMME, plant grade Elevation 583.0 ft is flooded for approximately 17 hr. Therefore, incident waves attacking the shoreline can be transmitted inland across the flooded plant grade. These transmitted wave heights depend on the available water depth above plant grade, the incident wave characteristics attacking the shoreline, the configuration of the shore barrier, and the location and configuration of other obstacles.

A rock shore barrier has been constructed in front of Fermi 2 along the shore between Plant Coordinate System Grid N6800 and N7800. The rock shore barrier crest elevation is 583 ft nominal; the toe elevation will be 572 ft nominal. For design wave considerations, a design toe elevation of 569.0 ft was used to allow for 3 ft of scour at the toe.

Transmitted wave heights (Reference 20) over the shore barrier are shown in Figure 2.4-17 for maximum and significant incident wave heights at the shore barrier. The incident water

depth at the shore barrier toe and the inland depth of water above a plant grade elevation of 583.0 ft are also indicated in Figure 2.4-17.

Using this inland depth of water caused by flooding of plant grade, a curve indicating the maximum wave height that can be supported over the flooded plant grade, without breaking, is presented in Figure 2.4-17. During the maximum flooding of plant grade, the maximum supported wave height is less than the transmitted wave heights. Therefore, the maximum supported wave height is the controlling factor for plant structures located more than a few hundred feet inland from the shoreline. The maximum inland supported wave heights for plant grade Elevation 583.0 and 580.0 ft are 3.0 and 5.4 ft, respectively. The actual site grade at a given location may vary from the reference elevation of 583.0 ft. However, the resultant difference in the hydrostatic pressure due to the difference of supported wave heights would be insignificant.

Waves that are transmitted over the shore barrier will attack the office service and radwaste buildings of Fermi 2. These buildings are not Category I structures and, therefore, could be damaged during the storm without causing a safety concern to the public.

Small waves can reach the Category I structures by traveling around the northerly and southerly ends of the shore barrier. Waves traveling around the ends of the shore barrier undergo several effects, including the following:

a. Breaking caused by the shallow depths of the flooded plant grade

b. Diffraction around the ends of the other plant structures

c. Reflection off plant structures before reaching the Category I structures

d. Reduction caused by plant grade bottom friction and side friction of obstructing structures.

The significant wave period of 7.7 sec will approach the plant sites from due east, while lower period waves can approach the northerly end of the shore barrier from 65° (N65°E), and possibly approach the southerly end from 110° (E20°S). Waves approaching the north end of the shore barrier will be reduced to the maximum inland support wave heights of 3.0 and 5.4 ft for plant grade Elevations 583.0 and 580.0 ft, respectively, in approaching Category I structures. Waves approaching the southerly end of the shore barrier will be reduced in height approaching Category I structures as a result of the maximum inland supported wave height and the protection provided by the office service and turbine buildings. Neglecting any reduction effects from protection provided by the office service and turbine buildings, waves approaching Category I structures from the south will be reduced to the maximum inland supported wave height of 3.0 ft for the plant grade elevation of 583.0 ft.

2.4.5.4.2.4. Wave Stability

In selecting the proper design wave for wave runup and wave forces against Category I structures, the wave period spectra must be considered since the significant wave period might not control. In calculating minimum wave periods, Equation 2.4-2 was used to determine the limiting wave steepness in shallow water (Reference 22).

$$H/L = 1/7 \tanh\left[\frac{2\pi d}{L}\right] \quad (2.4\text{-}2)$$

As mentioned in Subsection 2.4.5.4.2.3, waves attacking Category I structures are controlled by the available water depth over the flooded plant grade elevations. For plant grades with very flat slopes, the maximum supported wave height is approximately 0.78 times the water depth. The plant grade of Fermi 2 is Elevation 583 ft 0 in., and therefore a maximum wave height of 3.0 ft can be supported. Where the plant grade elevation is 580 ft 0 in., a maximum wave height of 5.4 ft can be supported. With the plant grade elevation changing from 580.0 ft to 583.0 ft in the vicinity of Grid N8000, it would be possible for either the 3.0-ft or the 5.4-ft wave to strike the north or east sides of Category I structures. Minimum wave periods calculated for wave heights of 3.0 ft and 5.4 ft are 3.4 sec and 4.5 sec, respectively. The maximum wave period of about 9 sec (Reference 22) is for a significant wave height of 7.8 ft and a significant wave period of 7.7 sec.

2.4.5.5. Resonance

Resonance generated by waves can be a problem in enclosed bays or harbors when the natural period of oscillation of the bay is equal to the period of the incident waves. However, the Fermi site is not located in an enclosed embayment. The full exposure of the site to Lake Erie during PMME conditions, plus the flat slopes surrounding the site area, result in a natural period of oscillation of the flooded area that is much greater than that of the incident shallow-water storm waves. Consequently, resonance is not a problem at the site during the PMME occurrence.

2.4.5.6. Runup

2.4.5.6.1. Flood Levels

Refer to Subsection 2.4.2.2 for a discussion of flood levels.

2.4.5.6.2. Maximum Runup Elevations

Maximum runup elevations on the exposed north faces of the reactor/auxiliary building and the RHR complex are 593.0 and 598.0 ft for the 3.0-ft and 5.4-ft waves, respectively. The maximum runup elevation on the exposed south faces of the reactor/ auxiliary building and the RHR complex, the exposed east face of the RHR complex, and the west face of the reactor/auxiliary building is 593.0 ft for the 3.0-ft wave. This wave could possibly reach the west face of the reactor/auxiliary building by reflection from the east face of the RHR complex. The east face of the reactor/auxiliary building is not exposed to waves and wave runup. The west face of the RHR complex is landward of the storm direction and not subject to waves and wave runup. As previously stated, no shore protection is required to preclude flooding of these structures.

2.4.5.6.3. Wave Forces

Maximum wave pressures and forces against Fermi 2 Category I structures can result from a 3.0-ft or possibly a 5.4-ft wave striking the north or east faces of Category I structures. These wave heights are the maximum supported wave heights for plant grade Elevations

583.0 and 580.0 ft. Wave pressures and thrusts against smooth vertical walls have been calculated from nonbreaking, broken, and breaking wave conditions. The wave periods have been varied from the minimum wave period to the maximum wave period. The instantaneous impact forces produced by waves breaking against a structure result in intense shock pressure with a duration in the range of 1/100 to 1/1000 sec. The intense pressures occur when a thin cushion is entrapped by waves breaking on a structure.

The breaking wave conditions are calculated from Minikin's formula. In adapting Minikin's formula, unrealistic results are predicted for very flat slopes (slopes fronting a vertical wall). Therefore, when the actual slope is flatter than 20:1 or even 10:1 (horizontal to vertical), pressures derived from a 20:1 or 10:1 slope should be used. Pressures and thrusts from breaking wave conditions were calculated for both slope conditions. Porous fill material, which can become completely saturated during flooded conditions, is placed from the top of slab elevation of the Category I structure to the plant grade elevation. Therefore, hydrostatic pressures against Category I structures are considered to the depth of the upper surface of the slab of both buildings.

Wave pressure and thrust results for the reactor/auxiliary building and the RHR complex are presented in Figures 2.4-18 and 2.4-19. Wave pressure distribution diagrams are presented in Figures 2.4-20 and 2.4-21. The critical static pressure and thrust occur under the broken wave conditions, whereas the critical dynamic pressure and thrust occur under the breaking wave conditions for an assumed slope of 20:1 and the minimum wave periods of 3.4 to 4.5 sec. All Fermi 2 Category I structures are designed to withstand these forces.

2.4.5.7. Protective Structures

The importance of the shore barrier in providing protection for Category I structures during the PMME has been greatly reduced from the originally approved concept for the following reasons:

a. Category I structures are not susceptible to flooding from storm surge and wave runup

b. Category I structures are largely protected by other plant facilities

c. Category I structures are not subject to damage from transmitted waves behind the barrier

d. Category I structures are not endangered by wave forces from 3.0-ft to 5.4-ft waves

e. Damage to the shore barrier will not enable waves larger than 5.4 ft to break against Category I structures since these structures are located a minimum distance of 800 ft inland from the shoreline. Safety-related structures that are located this distance away would remain safe during the extreme high stillwater levels of the PMME.

The shore barrier design and location are shown in Figure 2.4-22. The parameters used in the shore barrier design are discussed in detail in this section. The shore barrier ends are to be constructed on a side slope of 3:1 (horizontal to vertical) as compared to the design slope of 2:1 used for the shore barrier. The ends of the shore barrier rubble-mound structures are of

the same design as determined for the 2:1 slope. Criteria for construction of the multilayered barrier are shown in Figure
2.4-22. The ends have been flattened to a 3:1 slope to ensure that they can withstand conditions more severe than the design conditions.

A shore-barrier-slope-stability analysis was performed to deter-mine the factor of safety against sliding of the shore barrier, and it was concluded that the shore barrier has a sufficient factor of safety with regard to a sliding failure occurring at any soil layer. A report of this analysis was submitted to the NRC in July 1981.

The shore barrier, which allows for the possibility of 6 to 8 percent stone displacement during the PMME, extends from Grid N6800 to N7800 and preserves the integrity of the plant site fill placed to Elevation 583.0 ft.

The shore barrier, including the ends, consists of a rubble-mound structure using an armor cover of stone. A toe elevation of 572.0 ft, a crest elevation of 583.0 ft, and a lakeward-side slope of 2:1 (horizontal to vertical) were considered in its design. The design wave was based on the probable maximum storm event and a design shore barrier toe elevation of 569 ft, allowing for 3 ft of scour. Hudson's stability equation was used for determining the weights of armor units (Reference 21). Stability coefficients (K_D) listed in Reference 21 were used for significant wave conditions and are conservative values based on zero damage criteria for model studies. By allowing for some shore barrier damage (displacement of armor stones), a higher stability coefficient was used.

An armor cover was calculated using rough angular stone (density 165 lb/ft^3) placed on a 2:1 slope. Using a design toe elevation of 569.0 ft, the maximum significant breaking wave height (Figure 2.4-14) is found to be 12.2 ft during the probable maximum storm event. The possibility of some stone displacement (6 percent to 8 percent) was allowed for, with any displaced stones being replaced after the storm passed. A stability coefficient of 5.0 was used for two layers of stone placed randomly. This results in an armor layer 7.5 ft thick using 3.3-ton to 5-ton stone, as shown in Figure 2.4-22. The secondary layer is 3.5 ft thick with 600-lb to 1000-lb stone, while the filter layer is 1.5 ft thick, consisting of 30-lb to 50-lb stone. Below the filter layer is 1 ft of crushed rock (20 lb and under).

Where the plant grade elevation slopes from 580.0 to 583.0 ft, to the north of the Fermi 2 location, the slope is protected against the possibility of breaking 5.4-ft waves during the maximum stillwater level. Protection of the slope is achieved by lining it with suitable rock.

The NRC evaluated the as-built condition of the shore barrier and concluded that it met the requirements of General Design Criterion (GDC) 2 and was, therefore, acceptable on the basis that the inspection and maintenance program required by the Technical Requirements Manual provided reasonable assurance that the shore barrier would not be allowed to deteriorate significantly from its as-built configuration. The Technical Specifications require that the shore barrier be inspected on an annual basis and after major storms and seismic events exceeding operating-basis earthquake (OBE) intensity and be promptly restored to its prior condition in the event of any significant damage.

2.4.6. Probable Maximum Tsunami Flooding

The Fermi site is located in an area of the United States designated as having potentially minor seismic activity. Any tsunami activity in Lake Erie could only be generated by local seismic disturbances. Based on the history of the area, local seismic disturbances would result only in minor excitations in the lake. No tsunami has been recorded in Lake Erie; the only remotely similar phenomena observed have been low-amplitude seiches resulting from sudden barometric pressure differences. The low-amplitude seiches that could occur would be of negligible concern to the site.

2.4.7. Ice Flooding

Ice flooding is not a design basis at the Fermi site. The grade elevation of the plant site is at least 10 ft above the normal winter level of Lake Erie, and the emergency supply of water for cooling is not dependent upon natural bodies of water or the operation of intakes located where ice flooding could occur.

2.4.8. Cooling Water Canals and Reservoirs

2.4.8.1. Canals

A discharge canal is provided between the natural draft cooling towers and the circulating water reservoir. The canal is not part of a Category I system and is not safety related or necessary for the safe shutdown of the reactor.

2.4.8.2. Reservoirs

An open pond reservoir is provided as a collection basin from the natural draft cooling tower discharge to the circulating water pump house. The reservoir is not part of a Category I system and is not safety related or necessary for the safe shutdown of the reactor.

In addition, a reservoir is provided in the RHR complex. This is a Category I reservoir that is part of a closed cycle system that is not dependent upon natural bodies of water for makeup. The design basis for this complex in relation to water levels is described in Section 3.4.

2.4.9. Channel Diversions

The plant does not use water from channels; therefore, this subsection is not applicable.

2.4.10. Flooding Protection Requirements

All safety-related plant features are designed to withstand combinations of flood conditions and wave runup as discussed in Subsections 2.4.2.2 and 2.4.5.4. Protection of safety-related structures and components, including the effects of floods and waves, is discussed in Section 3.4 and Subsection 2.4.5.7.

2.4.11. Low Water Consideration

2.4.11.1. Low Flow in Rivers and Streams

Plant water sources are not related to the flow of rivers and streams in the area, except to the minor extent that these flows affect the general water level of Lake Erie.

2.4.11.2. Low Water Resulting From Surges, Seiches, or Tsunamis

2.4.11.2.1. Minimum Monthly Mean Lake Level

A summary of the historical minimum monthly mean lake levels was recorded by the U.S. Lake Survey during the period 1860 to 1973 and is presented in Figure 2.4-9. The minimum historic monthly mean lake level was reduced by approximately 40 percent of the recorded range of low water conditions (0.9) to give a minimum monthly mean design lake level of -1.5 ft below Low Water Datum.

2.4.11.2.2. Wind Setdown

Using the computer model prepared by Platzman (Reference 14 and Subsection 2.4.5.1), values were obtained for winds of varying speed from a westerly direction. Calculations based upon U.S. Weather Bureau data at Asheville, North Carolina, indicate that westerly winds of 70 mph sustained over a period of 6 hr would have a recurrence interval of one in 250 years. Using these values, the decrease in water level resulting from wind setdown at the site would be -9.2 ft (Elevation 561.3 ft).

Based upon probable maximum estimates of westerly winds furnished by the AEC, maximum wind setdown of the lake water level was calculated by Platzman's method (Reference 14) as -11.2 ft. The selected design wind setdown is -11.6 ft (Elevation 558.9 ft). This is identical to the calculated design PMME storm surge except with a minus instead of a plus sign.

2.4.11.2.3. Local Seiches and Tsunamis

For the same reasons as given in Subsections 2.4.5.2.3 and 2.4.6, no decrease in water level is assumed to occur from seiche and tsunami activity.

2.4.11.2.4. Design Level

Assuming that the effect of wind setdown occurs simultaneously with extreme minimum monthly lake levels, the resulting design stillwater level is Elevation -13.1 ft (Low Water Datum), or Elevation 557.4 ft.

The cooling water supply for safety-related systems is provided by the RHR complex, which contains its own water reservoir and is independent of ground water or lake-water level conditions. See Subsection 9.2.5 for a discussion of the RHR service water system.

2.4.11.3. Historical Low Water

The lowest observed monthly mean lake level during the period of record (1860 to 1973) was during February 1936, when Elevation -1.2 ft (Low Water Datum) was recorded. Low lake levels are generally recorded during the month of February. The most extreme setdown on

record (1897 to present) was -7.1 ft on March 22, 1955. This level was calculated from gage records obtained at Gibraltar and Toledo.

If coincident occurrence of the minimum historical lake level and setdown is assumed (-8.3 ft), a minimum probable low water elevation of 562.2 ft is obtained. The conservatism of the design values is realized by comparing these figures with the respective -1.5-ft and -11.6-ft values that were combined for the design level elevation of -13.1 ft.

2.4.11.4. Future Control

There is no future control anticipated for Lake Erie (Reference 23). Drainage improvements on Swan Creek have been made, but no additional controls are planned (Reference 24).

2.4.11.5. Plant Requirements

As described in Subsection 9.2.5, the cooling water supply for safety-related systems is provided by the RHR service water system, which contains its own water reservoir and is independent of ground- or lake-water supplies.

The main plant cooling water supply is provided by the circulating water pond (Subsection 10.4.5) and requires only makeup water from Lake Erie.

2.4.11.6. Heat Sink Dependability Requirements

The RHR complex contains the ultimate heat sink for Fermi 2, which is the RHR service water system. The RHR complex includes a man-made structure with a self-contained reservoir and is discussed in Subsection 9.2.5. This service water complex is independent of local water-level conditions.

2.4.12. Environmental Acceptance of Effluents

Discharge of liquid radwaste effluents is through a decant line into Lake Erie. The release point is indicated in Figure 2.1-5. Liquid effluent accidentally released at the surface from the plant eventually flows either eastward into Lake Erie or into the north lagoon after percolation downward through the crushed-rock fill. The configuration of the surface-area drainage pattern does not permit flow westward toward inland areas. Since the lagoon drains into the lake via Swan Creek, liquid surficial discharges would ultimately reach and be diluted by waters of Lake Erie. Any percolation into ground water ultimately reaches Lake Erie (Subsection 2.4.13). The locations and users of surface and ground water pertinent to effluent releases from the plant are provided in Subsections 2.4.1.2 and 2.4.13. The effects of plant effluent releases to Lake Erie were examined by calculating dilution factors at the Monroe intake and the Toledo intake.

Studies of the currents and dilution capacity of Lake Erie were made by Ayers (Reference 25) who found that except under ice-cover conditions there are two primary current patterns, northward and southward, with a velocity range from 0.1 to 0.3 mph. During ice-cover periods, the current is predominantly southerly with a velocity of less than 0.1 mph. The probable percentages of occurrence of the current patterns are 30 percent, southerly; 50

percent, northerly; and 20 percent, phase system. The duration of ice-cover ranges from 1 to 4 months.

Based on Ayers' measurements, dilution factors for the Monroe intake and the Toledo intake were estimated and are summarized in Table 2.4-6. The dilution factors were determined using the plant blowdown discharge line into Lake Erie as the effluent release point.

The annual average dilution factor was calculated on the basis of 40 percent (southerly) and 60 percent (northerly) current directions, with an ice-cover duration of 2 months occurring during southerly current conditions. Current velocities used in the calculations are 0.394 fps under ice-free conditions and 0.117 fps under ice-cover conditions. The worst condition for dilution factors is based on a southerly current under ice-cover conditions with a current velocity of 0.04 fps.

The subsurface diffusion of accidental releases of liquid radioactive effluents is considered in Subsection 2.4.13.

2.4.13. Ground Water

2.4.13.1. Description and Onsite Use

Ground water is not used as a source of water supply for the plant. Ground water features are subsequently described.

2.4.13.1.1. Regional Ground Water Features

The project area is located in the eastern lake section of the central lowlands physiographic province (Figure 2.5-1). Bedrock formations dip northwest into the Michigan Basin. They are generally covered by glacial drift deposits that vary considerably in thickness and composition. The bedrock topography at the base of the drift is irregular as a result of erosion and differential scouring by Pleistocene glaciation.

The drift deposits range from nearly impervious till to coarse channel deposits of gravel and boulders. To the northwest of the site, drift deposits occur that are sufficiently thick and permeable enough to allow development of ground water. To the south, soluble limestone and dolomite formations compose the principal aquifers. The distribution of these regional aquifers, as described by the USGS (Reference 26), is shown in Figure 2.4-23. Regional aquifers capable of furnishing public ground water supplies do not exist near the site because the bedrock formations are not highly pervious and contain poor quality water. The drift is thin and consists of nearly impervious till. Ground water conditions in Monroe County are described by Sherzer (Reference 27) and by Mozola (Reference 11).

Bordering Lake Erie and surrounding the site area are soils associated with former higher stages of Lake Erie. The soils are thin, generally organic, and do not serve as aquifers. The soil units are described in Subsection 2.5.1.1.2. Geologic units in the site region, principally the bedrock formations, are described in detail in Subsection 2.5.1.1.

2.4.13.1.2. Local Ground Water Features

In the site area, geologic units consist of bedrock formations that are overlain by thin and nearly impervious till and lacustrine deposits (Subsection 2.5.1.2). At the site, the lacustrine and till units have been partially excavated and replaced with crushed-rock fill (Subsection 2.4.1.1).

The till and lacustrine deposits are too thin and impervious to serve as aquifers. They are about 14 ft thick at the site. Descriptions of these deposits are given in Subsection 2.5.1.2.7.

The test borings explored the bedrock formations beneath the site to depths of 324.7 ft, penetrating the Bass Islands Group and part of the Salina Group. The formations dip slightly to the northwest (Subsection 2.5.1.2.3.2). The uppermost bedrock formation at the site is the Bass Islands Group; the upper surface of the Bass Islands is erosional and somewhat irregular. It is covered with till and lacustrine deposits less than 20 ft thick. At the site, the upper surface of the Bass Islands is about 550 ft elevation (Subsection 2.5.1.2.2) and exists to a depth of about l00 ft (Figure 2.5-15). It is directly below glacial drift in a 7-mile-wide band bordering Lake Erie (Figure 2.5-5). The Bass Islands Group consists of thin-bedded, fractured, locally vuggy, gray-brown dolomite, with carbonaceous shale partings. The formation is described in greater detail in Subsection 2.5.1.2.2. The Bass Islands Group comprises a confined aquifer at the site. During the exploration borings program, there was artesian flow from a number of borings penetrating the Bass Islands Group (Figures 2.5-24 through 2.5-56). Ground water in the Bass Islands Group is confined by the overlying till and lacustrine deposits. During construction dewatering, the ground water is drawn down below the confining layer.

Below the Bass Islands Group are fractured limestone and dolomite formations of the Salina Group. The Salina Group formations appear to comprise aquifers even in the argillaceous beds because test borings at the plant site encountered artesian flows from them.

Water quality was sampled at various zones. The water is highly mineralized. Sulfate content was similar in all formations. Results of the chemical analyses of the zones tested are shown in Table 2.5-16 and discussed in Subsection 2.5.1.2.4.

The aquifers receive recharge by infiltration of precipitation on higher ground areas west of the site as indicated by a mapping of the regional ground water level, shown in Figure 2.4-24. Because the ground water surface approximates the shape of the land surface, water apparently can percolate through the till. The map was prepared from water levels measured in wells completed within the Bass Islands dolomite. These well locations are shown in Figure 2.4-25. Water-level measurement data for the wells are presented in Table 2.4-7. The slope of the water level toward Lake Erie indicates that the lake comprises the ultimate sink for ground water flow.

The permeability data developed from pressure tests of borings at the Fermi site are described in Subsection 2.5.4.6. Of 29 tests in four borings, permeability varied from 210 to 2220 ft/yr. The average was 763 ft/yr. Because permeability is developed in rock joints and fractures, it can vary considerably from place to place.

Ground water is not a water supply source for the plant or any of its supporting facilities.

2.4.13.2. Sources

All municipal supplies within 25 miles of the site are from streams or lakes (Reference 28). In areas not served by municipal water systems, water supplies for domestic use are generally obtained from private wells. There are no industrial or municipal water wells in the site area (Reference 7). The network of private wells presently in use forms the source of water for domestic and livestock purposes in farms and homes west and north of the site, and for residences in the Stony Point area to the south, where the largest concentration of wells in the area occurs. The distribution of private water wells surrounding the site area is shown in Figure 2.4-26. This figure shows that there are about 4000 wells within 10 miles of the site. A survey of available drillers' records on approximately 400 wells in the site vicinity, filed at the Michigan Department of Natural Resources, shows that well depths generally do not exceed 70 ft. The wells are 4 to 6 in. in diameter, drilled into dolomite bedrock, and cased only through overburden soils into bedrock. Casings are uncemented, and the remainder of the hole below the casings is left open. Pumps are submersible or centrifugal (suction) type, having a capacity of about 10 gpm or less. The pumpage of water per well is probably on the order of 200 to 400 gal per day, typical of residential use. A certain amount of seasonal variation in water use can be expected because in summer months lawns and gardens are irrigated.

There has been virtually no long-term ground water level decline in the site area. The largest concentration of wells is in Stony Point. Pumping there may have lowered the water levels by 5 to 10 ft, on the basis of water levels reported on numerous drillers' logs since the 1940s. The radius of influence of pumping from these wells cannot be detected more than 1 mile away from Stony Point, on the basis of water-level data. Pumping from an onsite rock quarry operation in 1969-1972 caused a temporary lowering of water level. Pumping was terminated in June 1972 and the abandoned quarry was allowed to fill with ground water. The piezometric surface in the vicinity of the quarry returned to its normal level by the summer of 1973. The ground water level was monitored during the quarry dewatering and the data are shown in Table 2.4-7. Water level in the quarry is now approximately at land surface.

At the site, the confining layers have been stripped to permit the excavation for subgrade structures constructed in the aquifer. Backfill around the completed structures will not permit percolation into the aquifer at the site (Subsection 2.4.1.1).

The water use trend in the area is from ground water to surface water. The low transmissibility of the formation will not permit large-yielding water wells. Undesirable water quality is typical. As described in Subsection 2.5.1.2.9 and noted on boring logs, the ground water is high in sulfate content and hydrogen sulfide. Many neighboring communities, for example Woodland Beach and Berlin Township, have recently abandoned individual water wells in favor of a surface-water treatment-distribution system. Because surface water is available from nearby municipal systems for the communities in the area, the trend of increasing surface-water use and decreasing ground water use can be expected to continue in dense population areas. Isolated homesites, as on farms, will probably continue to use ground water.

Because of the trend toward decreasing use of ground water, it is improbable that any significant change in ground water gradient will occur from well pumping. The gradient is to the east, toward Lake Erie. There are no domestic wells downgradient from the site. If, for any reason, a reversal of ground water gradient from the site to the water wells were to occur, it would have to be for some reason other than pumping from the wells. This is true because, in order to create a gradient from the site to the water wells, the water level at the wells would have to be drawn down below their depth. It is therefore considered highly improbable that there will be any ground water condition in the future resulting in gradient reversal from the site toward the water wells.

The regional lakeward gradient is shown on the contour map of Figure 2.4-25. Water-level data used to prepare the map are shown in Table 2.4-7. Water levels at the site were depressed as a result of dewatering for Fermi 2 quarry operation. Prior to construction of Fermi 2, water flowed naturally from many of the borings in the area, as indicated on the boring logs in Figures 2.5-24 through 2.5-56. On the basis of the above-grade static level implied by these flows and water levels in wells in peripheral areas, it is suggested that ground water level at the site is normally above 575 ft.

Water levels in wells fluctuate seasonally, generally highest in spring and lowest in fall. Seasonal fluctuations are not related to Lake Erie fluctuations, although seasonal peaks are somewhat coincidental. The Lake Erie fluctuations are of lower magnitude (Subsection 2.4.2) than ground water fluctuations. It is suggested that the fluctuations coincide because both water bodies respond to the same influences of recharge and evapotranspiration. Water-level fluctuations in the site vicinity since 1970 are provided by the data in Table 2.4-7.

The nearest government agency observation well is approximately 20 miles to the west, in the Dundee area. It is monitored by the USGS. Because the well is completed in glacial drift, water-level fluctuations in the well cannot be considered representative of water-level fluctuations that would occur in the bedrock formation wells in the site area.

Flow rates within the aquifer are highly variable, owing to the fractured and jointed nature of the bedrock. The width, density, and directional pattern of openings can vary from place to place, as indicated by exposures of rock in excavations of the Fermi 2 site and in the onsite rock quarry to the south. An average velocity of flow in the bedrock aquifer is derived on the following basis:

Porosity, n = 0.01, conservatively assumed (Reference 29)

Permeability, k = 2 ft/day, from tests in borings

Hydraulic gradient, $I = \frac{3\text{ ft}}{2{,}500\text{ ft}} = 0.0012$, determined between wells 17M2 and 17Q1 (12/31/1973)

Velocity, V = kI/n = 0.24 ft/day

It is noted that the natural water-level gradient at the site is not available owing to construction dewatering at Fermi 2.

2.4.13.3. Accident Effects

Ground water conditions of the site (Subsection 2.4.13.1) consist of a bedrock aquifer confined under artesian pressure beneath a cap of relatively impervious glacial deposits. Under natural conditions, the ground water gradient is toward Lake Erie. Ground water moves in this direction and eventually discharges into the lake by moving upward through till and lake-bottom sediments.

In the unlikely event of an earthquake, minor cracking in the walls of at least the subgrade portion of the radwaste building structure could occur. The radwaste liquid storage tanks could also undergo stress cracking and leaking to allow fluid flow between the interior of the structure and the surrounding earth. Initially, liquid would be retained within the structure and diluted by inflowing ground water from the dolomite aquifer in contact with the structure. There would be a slow inflow of ground water and the water level inside the structure would rise until it attained the elevation of the piezometric level of the aquifer, approximately Elevation 575.0 ft. At this time, the radioactive material will have been diluted 10:1 or greater.

The time required to fill the structure would be on the order of 3 to 4 weeks. This length of time is determined on the basis of the following information:

a. During construction dewatering of the reactor building basement, pumping was stopped overnight and on weekends. The excavation became flooded up to 3 ft as a result of inflowing ground water. On one such occasion, the water-level rise in the excavation was measured. The rate of rise was 0.0281 ft/hr

b. It is assumed that this same rate of rise could occur in the radwaste building excavation, but adjusted to account for the space occupied by masonry and equipment, which is approximately one-third of the total floor area. The adjusted rate of rise is somewhat higher, almost 0.042 ft/hr

c. The rate of rise decreases continuously as the water level in the structure approaches ground water level. The assumption of a steady rate of water level rise of 0.042 ft/hr is therefore conservative.

During the 3- to 4-week period during which water is rising in the structure, equipment can be mobilized for pumping, storage, processing, and disposal of radioactive material.

If the structure is allowed to fill completely, diluted material would move into and through the aquifer at the same rate of flow and direction of movement as the existing ground water in the aquifer. The direction of movement would be to the east at a rate of 0.24 ft/day (Subsection 2.4.13.2).

The length of time required to travel the 460-ft distance from the structure to the Lake Erie shoreline is 1920 days. By this time, the specific activity of the radioactive material will have been below the limits set forth in 10 CFR 20. (For details of this accident analysis, see Subsection 15.7.3.)

For a discussion of flood protection of the onsite storage building, see Subsection 11.7.2.2.5.

2.4.13.4. Monitoring and Safeguard Requirements

It was demonstrated in Subsection 2.4.13.2 that no water wells are located downgradient from the site. As part of the operational radiological environmental monitoring program, Edison will measure the water level monthly in existing observation wells. The comparison of the data will show flow reversal if it occurs. Should a reversal in flow occur, grab samples would be taken and analyzed for gross beta and gamma isotopes if a path is available from the plant to the ground water. Results would be reported in accordance with the requirements of the Technical Specifications 5.6.2 and 5.6.3.

Under accident conditions, postulated in Subsection 2.4.13.3, monitoring wells will be drilled between the affected structures and the Lake Erie shoreline to monitor subsurface travel and dispersion of radioactive material. Exploratory drilling experience at the Fermi site indicates that truck-mounted drilling rigs are available from Detroit and Toledo and that an observation well could be drilled within several days.

2.4.13.5. Design Bases for Subsurface Hydrostatic Loadings

As described in Subsection 2.4.13.2, the natural ground water level at the site is on the order of 575 ft. As a conservative value for computing normal subsurface hydrostatic loadings, the ground water level is assumed to be 576.0 ft.

Because of the ground-level conditions, construction dewatering is necessary during all major building excavations. In the Fermi 2 construction, dewatering was done by sump pumps placed in the excavations. At the reactor building, grout curtains were installed to minimize ground water inflow and to prevent seepage that would cause falling rock from the walls of the excavations. The Fermi 2 reactor building excavation is 204 by 154 ft, with floor elevations of 540.0 and 551.0 ft.

Bedrock beneath the structure is dolomite, and was pressure grouted for added strength. The dewatering does not affect the structural integrity of the rock. All major safety-related structures have their foundations on bedrock and not within the overburden soils or drift (Subsection 2.5.4.11).

Water supply wells will not be used at the facility.

2.4.14. Technical Specifications and Emergency Operation Requirements

Fermi 2, together with its associated safety-related facilities, is designed to function in a safe manner despite the occurrence of any of the adverse hydrologic events previously discussed. These events have been postulated to occur in appropriate combinations, and such provisions for the safe operation of the plant have been incorporated into the design.

2.4.14.1. Flooding

The probable maximum water levels in Swan Creek resulting from precipitation or flood are discussed in Subsection 2.4.3. These levels are less than those anticipated from the probable maximum surge on Lake Erie.

2.4.14.2. Dam Failures

Potential dam failures are discussed in Subsection 2.4.4. It has been found that there are no regulatory structures on Swan Creek. In addition, there are no dams on other streams and rivers in southeastern Michigan, the failure of which would affect water levels in Lake Erie along the plant shoreline.

2.4.14.3. Surge and Seiche Flooding

The PMME is caused by storm surge. This event, discussed in Subsection 2.4.5, causes a stillwater level at the site of 586.9 ft, or 3.9 ft above plant grade elevation. As described, the Category I structures are designed for the PMME flood level plus runup from small waves generated on the flooded site. The openings in the structures are watertight and designed for the high-water levels.

The water levels associated with the seiche, discussed in Subsection 2.4.5, have been found to be less than the storm surge.

2.4.14.4. Tsunami

Tsunami is discussed in Subsection 2.4.6. Water levels associated with this event have been found to be less than for the storm surge.

2.4.14.5. Ice Flooding

Ice flooding is discussed in Subsection 2.4.7.

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TABLE 2.4-1 ESTIMATED DISCHARGE FREQUENCY - SWAN CREEK

Recurrence Interval (years)	Maximum Discharge (ft^3/sec)
2	2250
5	3500
10	4500
20	5800
50	7700
100	9300

TABLE 2.4-2 SYNTHESIZED LOCAL MAXIMUM PRECIPITATION[a]

Time (hr)	Cumulative Rainfall (in.)	Incremental Rainfall (in.)
1/4	4.9	4.9
1/2	7.0	2.1
3/4	8.8	1.8
1	10.2	1.4
2	14.3	4.1
3	18.0	3.7
4	21.3	3.3
5	24.2	2.9
6	26.9	2.7
12	29.2	2.3
18	31.0	1.8
24	32.4	1.4
30	33.2	0.8
36	33.8	0.6
42	34.3	0.5
48	34.7	0.4

[a] Data from Reference 5.

TABLE 2.4-3 SYNTHESIZED PROBABLE MAXIMUM PRECIPITATION FOR THE SWAN CREEK WATERSHED[a,b]

Time (hr)	Maxima for Durations Indicated: Cumulative Rainfall (in.)	Maxima for Durations Indicated: Incremental Rainfall (in.)	Increments of Storm Sequence (2-hr periods)
2	10.7	10.7	0.2
4	16.0	5.3	0.2
6	20.2	4.2	0.2
8	21.4	1.2	0.2
10	22.0	0.6	0.2
12	22.5	0.5	0.2
14	23.0	0.5	0.2
16	23.4	0.4	0.2
18	23.8	0.4	0.2
20	24.2	0.4	0.2
22	24.5	0.3	0.3
24	24.8	0.3	0.3
26	25.1	0.3	0.3
28	25.4	0.3	0.3
30	25.6	0.2	0.4
32	25.8	0.2	0.5
34	26.0	0.2	0.6
36	26.2	0.2	1.2
38	26.4	0.2	5.3
40	26.6	0.2	10.7
42	26.8	0.2	4.2
44	27.0	0.2	0.5
46	27.2	0.2	0.4
48	27.4	0.2	0.4

[a] Drainage area 109 square miles.
[b] Data from Reference 5.

TABLE 2.4-4 ESTIMATED PRECIPITATION LOSSES AND RUNOFF, PROBABLE MAXIMUM FLOOD, SWAN CREEK[a]

Time (hr)	Unit Hydrograph (ft^3 /sec)	PMP	Loss	Runoff	Surface Runoff From Rainfall Excess (ft^3 /sec)	Base Flow (ft^3 /sec)	Total Discharge (ft^3 /sec)
0		0	0	0	0	100	100
2	410	0.2	0.2	0	0	100	100
4	1070	0.2	0.2	0	0	100	100
6	1860	0.2	0.2	0	0	100	100
8	2640	0.2	0.04	0.16	66	100	166
10	3420	0.2	0.04	0.16	236	100	336
12	4000	0.2	0.04	0.16	534	100	634
14	3820	0.2	0.04	0.16	957	100	1,057
16	3440	0.2	0.04	0.16	1,504	100	1,604
18	3010	0.2	0.04	0.16	2,144	100	2,244
20	2520	0.2	0.04	0.16	2,755	100	2,855
22	2060	0.3	0.04	0.26	3,347	100	3,447
24	1710	0.3	0.04	0.26	3,935	100	4,035
26	1410	0.3	0.04	0.26	4,524	100	4,624
28	1160	0.3	0.04	0.26	5,188	100	5,218
30	900	0.4	0.04	0.36	5,775	100	5,875
32	700	0.5	0.04	0.46	6,548	100	6,648
34	510	0.6	0.04	0.56	7,450	100	7,550
36	350	1.2	0.04	1.16	8,741	100	8,841
38	160	5.3	0.04	5.26	12,269	100	12,369
40	22	10.7	0.04	10.66	21,325	100	21,425
42	0	4.2	0.04	4.16	35,034	100	35,134
44		0.4	0.04	0.46	50,805	100	50,905
46		0.4	0.04	0.36	66,564	100	66,664
48		0.4	0.04	0.36	80,588	100	80,688
50					88,432	100	88,532

[a] Drainage area 109 square miles.

TABLE 2.4-5 U.S. ARMY CORPS OF ENGINEERS UNIT HYDROGRAPHS

Basin	Station	Drainage Area (mi^2)	q_p	t_p	C_p 640	C_t	$(LL_{ca})^{0.3}$	L	L_{ca}	t_r (hr)
Swan Creek[a]	Mouth, Michigan	109	36.7	12.3	451	2	6.14	25.4	16.67	2
Cedar River	East Lansing, Michigan	355	7.6	36.5	279	5.1	7.1	37	18	6
Sandusky River	Bucyrus, Ohio	89.8	27.1	21.0	569	3.39	6.2	27.5	16.3	6
Sebewaing River	Sebewaing, Michigan	105	28.46	11.0	313	2.50	4.44	16	9	6
Juscarawas River	Massillon, Ohio	507	8.06	44.4	358	6.34	7.0	41.0	16.0	6
Clinton River	Mt. Clemens, Michigan	733	17.5	22.2	441	3.81	6.62	32	17	6
Grand River	Lansing, Michigan	1230	6.8	38.5	260	3.4	11.2	75	42	6

[a] Synthetic unit hydrograph.

TABLE 2.4-6 DILUTION FACTOR ESTIMATES - LAKE ERIE INTAKES

	Normal Conditions					
	South Current		North Current		Annual	Worst
Location	Ice-Free	Ice-Cover	Ice-Free	Ice-Cover	Average	Condition
Monroe intake	320	290	1.6×10^{11}	1.0×10^{10}	770	26
Toledo intake	1.6×10^{16}	9.0×10^{12}	3.1×10^{25}	1.1×10^{22}	5.4×10^{13}	4.3×10^{5}

TABLE 2.4-7 WATER WELL DATA[a]

Map Reference Number	Well Number	Depth (ft)	Elevation of Water Level (ft)	Date
R1	5S/8E-36R1[b]	77	594.0	9/9/64
			597.6	4/28/72
D1	5S/9E-2D1[b]	33	590.0	5/20/65
			588.11	4/28/72
J1	6S/9E-11J1[b]	--	581.22	2/3/72
K1	6S/9E-13K1	--	577.02	12/29/70
			577.25	12/30/70
			576.68	10/22/71
C1	6S/9E-23C1	35	580.74	2/3/72
			583.0	11/13/54
K1	6S/9E-23K1	95	572.0	11/24/69
			570.64	9/8/70
Q1[c]	6S/9E-23Q1	76	572.0	11/6/69
			575.4	9/8/70
			574.65	10/27/70
			576.39	12/29/70
			575.8	2/26/71
			577.0	3/26/71
			576.25	4/30/71
			576.3	5/28/71
			574.8	7/2/71
			573.0	7/30/71
			572.8	8/24/71
			573.52	10/22/71
			572.3	10/30/71
			579.13	4/28/72
C1	6S/9E-24C1	--	576.87	12/29/70
Q1[c]	6S/9E-24Q1	50	575.0	9/19/69
			574.76	9/8/70
			573.84	10/27/70
			575.97	12/29/70
			573.4	11/5/71

TABLE 2.4-7 WATER WELL DATA[a]

Map Reference Number	Well Number	Depth (ft)	Elevation of Water Level (ft)	Date
			573.4	12/3/71
			574.4	1/7/72
			575.4	2/4/72
			576.1	3/3/72
			579.8	4/7/72
			580.5	4/21/72
			580.73	4/29/72
			582.15	5/26/72
			578.57	6/23/72
			578.23	7/7/72
			577.73	8/23/72
			578.57	10/6/72
			581.90	11/24/72
			582.07	12/29/72
Q2	6S/9E-24Q2	70	571.0	11/6/53
Q3	6S/9E-24Q3	65	577.0	6/13/53
R1	6S/9E-24R1	127.5	577.0	3/27/51
L1	6S/9E-25L1	32	568.0	8/2/56
L2	6S/9E-25L2	45	572.0	7/9/52
L3	6S/9E-25L3	41.5	570.0	4/28/50
L4	6S/9E-25L4	50.5	565.0	7/3/50
L5	6S/9E-25L5	28.5	572.0	6/17/53
			575.04	2/3/72
M1	6S/9E-25M1	49.5	574.0	4/17/53
M1A	6S/9E-25M1A	37	570.0	10/18/55
M2	6S/9E-25M2	39	575.0	4/12/48
	6S/9E-35H1	34.5	569.0	1/20/49
J1	6S/10E-6J1[b]	52	575.0	8/31/63
Q1	6S/10E-6Q1[b]	55	570.0	10/17/53
Q2	6S/10E-6Q2[b]	56.5	575.0	7/3/47
A1	6S/10E-7A1[b]	55	576.0	9/18/53
A2	6S/10E-7A2[b]	116	570.0	12/12/69

TABLE 2.4-7 WATER WELL DATA[a]

Map Reference Number	Well Number	Depth (ft)	Elevation of Water Level (ft)	Date
			570.7	2/3/72
H1	6S/10E-7H1[b]	52	567.0	6/12/56
K1	6S/10E-7K1[b]	67	576.0	6/6/68
L1	6S/10E-7L1[b]	35	572.0	7/1/50
J1	6S/10E-8J1[b]	49	575.0	12/21/55
K1	6S/10E-8K1[b]	36	571.0	11/26/57
R1	6S/10E-8R1[b]	51	571.0	1/30/66
			570.63	9/8/70
			570.03	2/3/72
B1	6S/10E-16B1[b]	52	572.0	
C1	6S/10E-16C1	49	570.0	6/25/54
F1	6S/10E-17F1	59	562.0	2/17/64
			568.91	9/8/70
M2	6S/10E-17M2	--	567.59	10/27/70
			571.75	2/3/72
P1[c]	6S/10E-18P1	60	572.1	9/8/70
			571.84	12/30/70
			576.3	2/26/71
			576.6	1/26/71
			573.2	5/28/71
18P1[c]	6S/10E-19P1	--	574.0	7/2/71
			575.0	7/29/71
			573.25	8/27/71
			573.30	9/24/71
			573.30	10/30/71
			571.2	12/3/71
			573.5	1/7/72
			573.6	2/4/72
			574.0	3/3/72
			577.3	4/7/72
			578.3	4/21/72
			576.67	4/29/72

TABLE 2.4-7 WATER WELL DATA[a]

Map Reference Number	Well Number	Depth (ft)	Elevation of Water Level (ft)	Date
			579.00	5/26/72
			576.92	6/23/72
			576.17	7/7/72
			573.50	8/25/72
			576.58	10/6/72
			581.17	11/24/72
			581.50	12/29/72
R1	6S/10E-18R1	80	573.49	9/8/70
			569.24	10/27/70
			569.56	12/29/70
B1	6S/10E-19B1	65	577.0	12/22/64
B2	6S/10E-19B2	65	583.0	2/17/69
			576.86	9/8/70
			571.86	10/27/70
			568.94	12/29/70
			583.0	2/17/69
			576.42	9/8/70
			571.42	10/27/70
			568.3	12/29/70
			571.33	8/6/71
			570.26	8/27/71
			570.21	9/24/71
			570.14	10/30/71
			570.94	12/10/71
			570.94	1/7/72
			571.84	2/4/72
			572.34	3/3/72
			575.02	4/7/72
			578.19	4/21/72
			576.69	4/29/72
			576.76	5/26/72
			574.69	6/23/72

TABLE 2.4-7 WATER WELL DATA[a]

Map Reference Number	Well Number	Depth (ft)	Elevation of Water Level (ft)	Date
			573.69	7/7/72
			573.94	10/6/72
			579.11	11/24/72
B3	6S/10E-19B3	45	581.0	10/30/53
G1	6S/10E-19G1	--	591.0	3/2/56
H1[c]	6S/10E-19H1	--	570.7	5/12/71
			570.4	6/1/71
			570.75	7/2/71
			570.32	8/2/71
			570.21	8/27/71
			570.57	10/1/71
			569.8	11/5/71
			569.5	12/3/71
			570.25	12/23/71
			572.0	1/31/72
			571.3	2/25/72
			573.0	3/14/72
			574.4	4/7/72
			578.0	4/21/72
			576.67	4/29/72
			575.58	5/26/72
			573.25	6/23/72
			572.50	7/7/72
			570.67	8/25/72
			572.67	10/6/72
			578.17	11/24/72
			578.92	12/29/72
M1	6S/10E-19M1	56	580.0	5/17/68
			570.03	9/8/70
			572.36	2/3/72
M2	6S/10E-19M2	40.5	580.0	12/8/45
M3	6S/10E-19M3	31	582.0	4/12/49

TABLE 2.4-7 WATER WELL DATA[a]

Map Reference Number	Well Number	Depth (ft)	Elevation of Water Level (ft)	Date
P1	6S/10E-19P1	58	569.0	10/6/64
R1	6S/10E-19R1	45	566.72	9/8/70
			573.94	4/28/72
P1[c]	6S/10E-20P1	84	568.0	3/18/70
			568.0	4/1/70
			567.3	5/6/70
			559.8	8/10/70
			562.2	8/19/70
			563.58	3/1/71
			565.38	4/1/71
			562.58	5/3/71
			554.48	6/1/71
			548.38	7/1/71
			544.78	7/23/71
			Destroyed	--
P2[c]	6S/10E-20P2	--	568.0	3/18/70
			567.2	5/6/70
			564.3	6/25/70
			563.9	7/30/70
			563.8	8/18/70
			566.92	3/1/71
			567.62	4/1/71
			565.92	5/3/71
			564.52	6/1/71
			559.12	7/1/71
			556.77	8/2/71
			552.02	8/27/71
			551.81	10/1/71
			550.94	11/5/71
			549.61	12/3/71
			549.14	12/23/71
E1	6S/10E-20E1	62	583.0	10/27/70

TABLE 2.4-7 WATER WELL DATA[a]

Map Reference Number	Well Number	Depth (ft)	Elevation of Water Level (ft)	Date
			585.18	4/28/72
E2	6S/10E-20E2	--	580.51	12/29/70
N1	6S/10E-20N1	53.5	565.0	5/26/50
C1	6S/10E-28C1	58	569.0	12/12/50
D1	6S/10E-28D1	39	568.19	10/22/71
D2	6S/10E-28D2	51.5	571.0	3/12/51
E1[c]	6S/10E-28E1	--	567.97	9/8/70
			567.88	10/27/70
			569.84	12/29/70
			571.5	2/26/71
			572.1	3/26/71
			571.75	4/30/71
			570.4	5/28/71
			568.5	7/2/71
			566.0	7/30/71
			566.17	8/27/71
			565.82	9/24/71
			565.9	10/30/71
			566.17	12/3/71
			567.5	1/7/72
			569.3	2/4/72
			570.84	3/3/72
			572.1	4/7/72
			572.8	4/21/72
			572.42	4/29/72
			571.50	5/26/72
			570.00	6/23/72
			569.58	7/7/72
			569.17	8/25/72
			570.92	10/6/72
			573.00	11/24/72
			573.42	12/29/72

TABLE 2.4-7 WATER WELL DATA[a]

Map Reference Number	Well Number	Depth (ft)	Elevation of Water Level (ft)	Date
E2	6S/10E-28E2	74.5	574.5	6/30/51
E3	6S/10E-28E3	43	577.0	5/1/56
E4	6S/10E-28E4	56.5	575.0	4/19/52
E5	6S/10E-28E5	51	572.0	7/28/65
E6	6S/10E-28E6	--	568.8	10/22/71
E7	6S/10E-28E7	--	569.4	10/22/71
			576.4	5/1/72
F1	6S/10E-28F1	68	573.0	11/20/67
			571.81	10/22/71
M1	6S/10E-28M1	68	572.0	5/17/49
A1	6S/10E-29A1	--	566.52	10/22/71
			570.65	4/28/72
B1[c]	6S/10E-29B1	--	567.45	7/1/70
			567.42	8/3/70
			566.22	9/1/70
			566.37	10/1/70
			566.87	11/2/70
			567.07	12/2/70
			567.17	1/4/71
			566.6	2/1/71
			568.57	3/1/71
			569.57	4/1/71
			568.43	5/3/71
			567.87	6/1/71
			565.97	7/1/71
			564.82	8/2/71
			564.15	8/27/71
			564.15	10/1/71
			563.57	11/5/71
			563.57	12/3/71
			563.77	12/23/71
			564.57	1/31/72

TABLE 2.4-7 WATER WELL DATA[a]

Map Reference Number	Well Number	Depth (ft)	Elevation of Water Level (ft)	Date
			563.87	2/25/72
			564.37	3/14/72
			565.27	4/7/72
			566.24	4/21/72
			566.40	4/29/72
			567.07	5/26/72
			564.99	6/23/72
			564.90	7/7/72
			566.24	8/25/72
			567.07	10/6/72
			569.74	11/24/72
			570.07	12/29/72
D1	6S/10E-29D1	28.5	570.0	10/2/54
			563.25	10/22/71
			567.45	4/28/72
E1	6S/10E-29E1	38.5	572.0	7/16/53
E2	6S/10E-29E2	31	567.0	8/31/55
E3	6S/10E-29E3	60.5	572.0	7/13/62
E4	6S/10E-29E4	40	572.2	1970
			562.4	10/22/71
H1	6S/10E-29H1	39	571.0	
H2	6S/10E-29H2	38.5	569.0	10/15/47
J1	6S/10E-29J1	37	570.0	5/27/60
J2	6S/10E-29J2	35	567.0	6/4/56
			570.55	2/3/72
J3	6S/10E-29J3	35	572.0	1/8/53
J4	6S/10E-29J4	74	566.0	11/18/52
J5	6S/10E-29J5	46	568.0	7/25/64
J6	6S/10E-29J6	40	572.0	6/2/52
J7	6S/10E-29J7	45	571.0	6/13/53
J8	6S/10E-29J8	28	572.0	4/12/49
J9	6S/10E-29J9	38	570.0	5/13/50

TABLE 2.4-7 WATER WELL DATA[a]

Map Reference Number	Well Number	Depth (ft)	Elevation of Water Level (ft)	Date
J10	6S/10E-29J10	31	570.0	7/29/53
J11	6S/10E-29J11	36	572.0	6/14/57
K1	6S/10E-29K1	30	575.0	3/19/52
K2	6S/10E-29K2	47	573.0	6/7/63
Q1	6S/10E-29Q1	40	566.0	
R1	6S/10E-29R1	30	573.0	4/18/57
R2	6S/10E-29R2	50	564.0	11/16/54
B1	6S/10E-30B1	60	569.0	10/7/68
C1	6S/10E-30C1	40	569.0	11/26/63
			568.93	2/3/72
E1	6S/10E-30E1	29	571.0	8/8/45
H1	6S/10E-30H1	42.5	570.0	9/18/65
H2	6S/10E-30H2	49	572.0	10/28/57
A1	6S/10E-32A1	49	570.0	6/7/56
A2	6S/10E-32A2	41.5	575.0	6/11/51
P2[c]	6S/10E-20P2		546.94	1/31/72
			547.14	2/25/72
			540.34	3/14/72
			537.99	4/7/72
			540.77	4/21/72
			541.86	4/29/72
			542.94	5/26/72
			539.11	6/23/72
			540.44	7/7/72
			552.86	8/25/72
			557.19	10/6/72
			561.52	11/24/72
			564.69	12/29/72
P3	6S/10E-20P3	62	576.0	12/15/65
			551.55	7/25/72
E1[c]	6S/10E-21E1	42	557.91	7/1/70
			559.59	8/3/70

TABLE 2.4-7 WATER WELL DATA[a]

Map Reference Number	Well Number	Depth (ft)	Elevation of Water Level (ft)	Date
			555.02	9/1/70
			555.74	10/1/70
			556.74	11/2/70
			556.60	12/2/70
			556.94	1/4/71
			556.1	2/1/71
			557.14	3/1/71
			556.94	4/1/71
			555.49	5/3/71
			556.54	6/1/71
			555.94	7/1/71
			555.99	8/2/71
			556.53	8/28/71
			557.12	10/1/71
			556.24	11/5/71
			556.24	12/3/71
			556.64	12/23/71
			558.14	1/31/72
			559.44	2/25/72
			559.64	3/14/72
			562.16	4/7/72
			562.99	4/21/72
			561.91	4/29/72
			561.99	5/26/72
			564.16	6/23/72
			563.99	7/7/72
			560.32	8/25/72
			560.37	10/6/72
			560.91	11/24/72
			563.74	12/29/72

TABLE 2.4-7 WATER WELL DATA[a]

Map Reference Number	Well Number	Depth (ft)	Elevation of Water Level (ft)	Date

[a] Shown in Figure 2.4-25.

[b] Not shown in Figure 2.4-25.

[c] Monitor wells are underlined.

Explanation of well numbering system:

The well locations are identifiable by the well number. The well numbering system, which is commonly used by water resource agencies, including the U.S. Geological Survey, designates the location of the well within a 40-acre parcel of land. The standard one-square-mile section is subdivided into 40-acre parcels as follows:

D	C	B	A
E	F	G	H
M	L	K	J
N	P	Q	R

As an example, suppose a given well is located as follows:

a. Township 7 South

b. Range 10 East

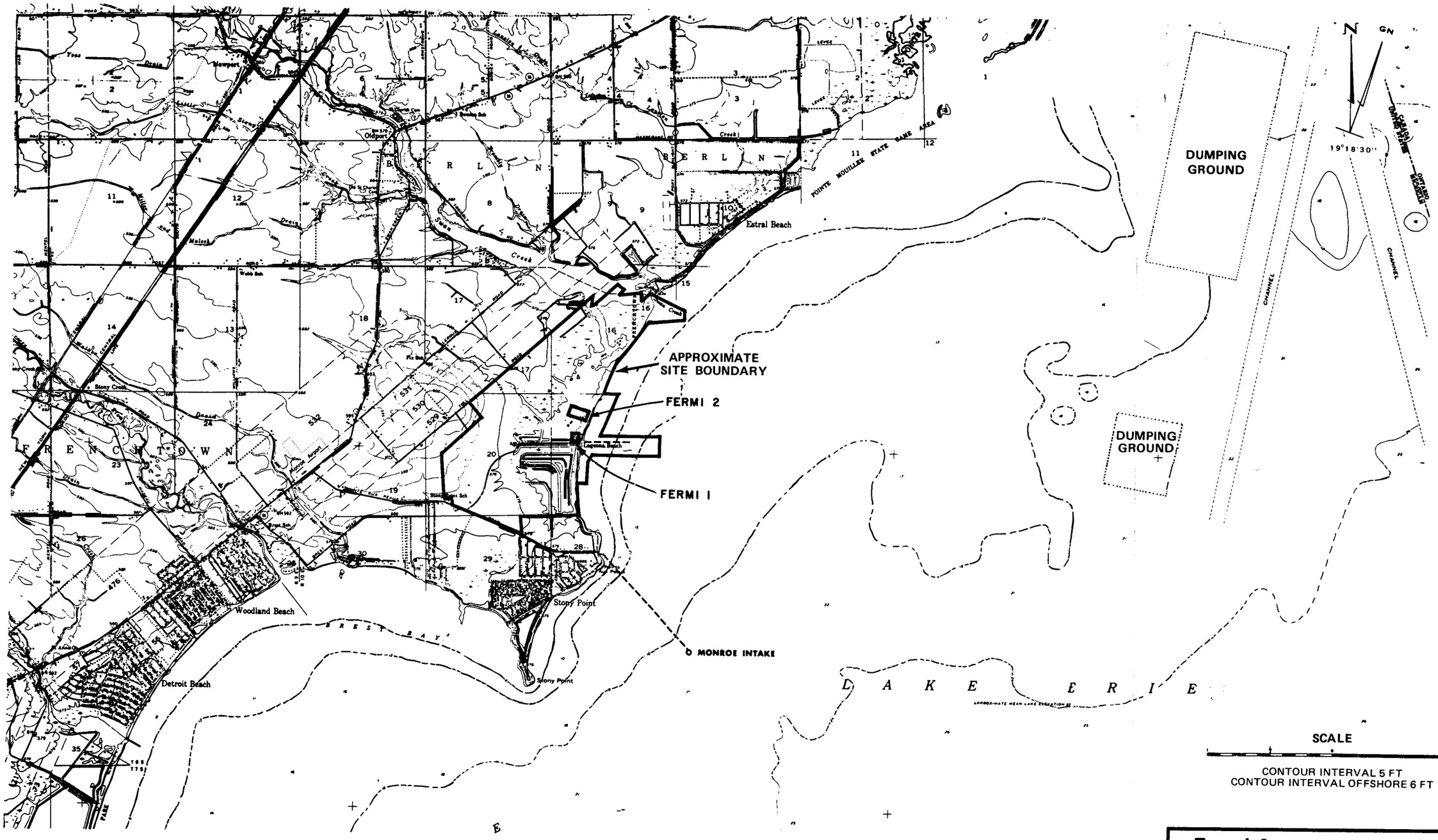
c. Section 32

d. northeast corner.

That well would be given the number, 7S/10E-32A1.

The number 1 following the letter A indicates that this is the first well inventoried in the 40-acre parcel lettered A.

All the wells within the immediate vicinity of the site are shown in Figure 2.4-25. These wells are identified and located by the last two digits of the previously described well numbering system and listed under the heading, "MAP Reference Number."



REFERENCE:
THIS MAP WAS PREPARED FROM PORTIONS OF THE FOLLOWING U.S.G.S. QUADRANGLES: ESTRAL BEACH, MICH., 1942, STONY POINT, MICH., 1952, ROCKWOOD, MICH., 1952, AND FLAT ROCK, MICH., 1952.

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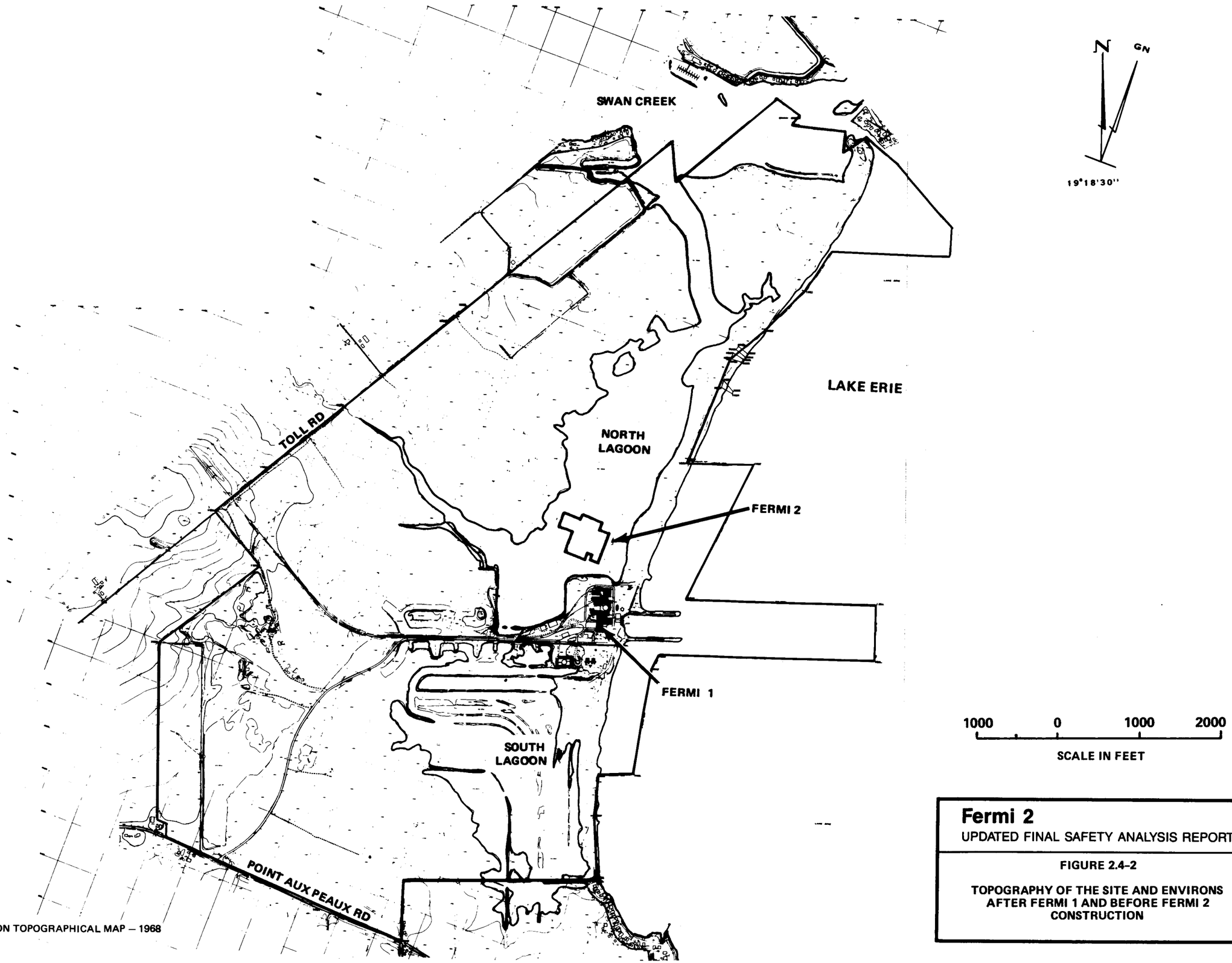
FIGURE 2.4-1

SITE VICINITY MAP



REFERENCE:
PORTION OF THE DETROIT EDISON TOPOGRAPHICAL MAP – 1968

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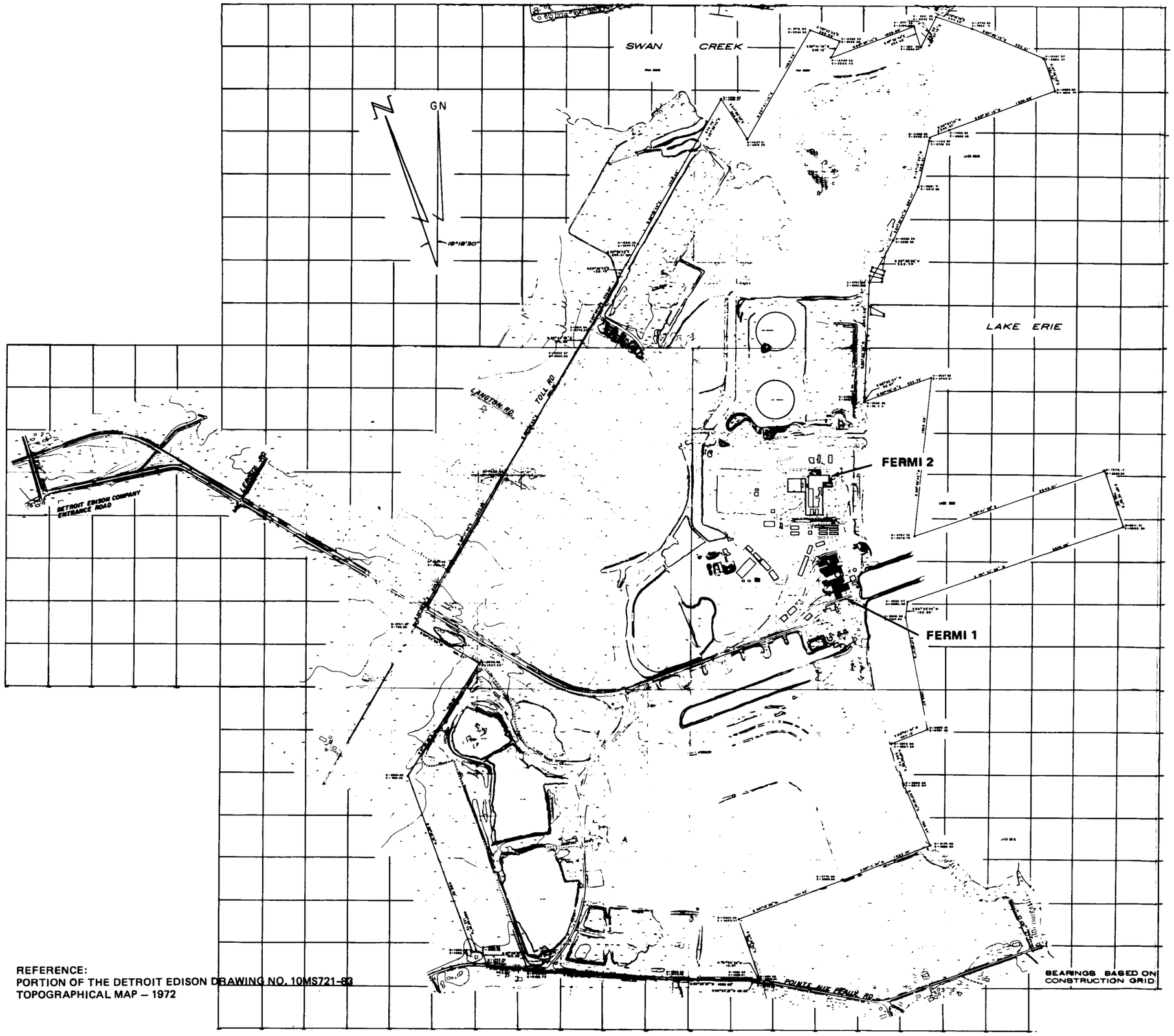
FIGURE 2.4-2

TOPOGRAPHY OF THE SITE AND ENVIRONS AFTER FERMI 1 AND BEFORE FERMI 2 CONSTRUCTION



1000 0 1000 2000

SCALE IN FEET

REFERENCE:
PORTION OF THE DETROIT EDISON DRAWING NO. 10MS721-83
TOPOGRAPHICAL MAP – 1972

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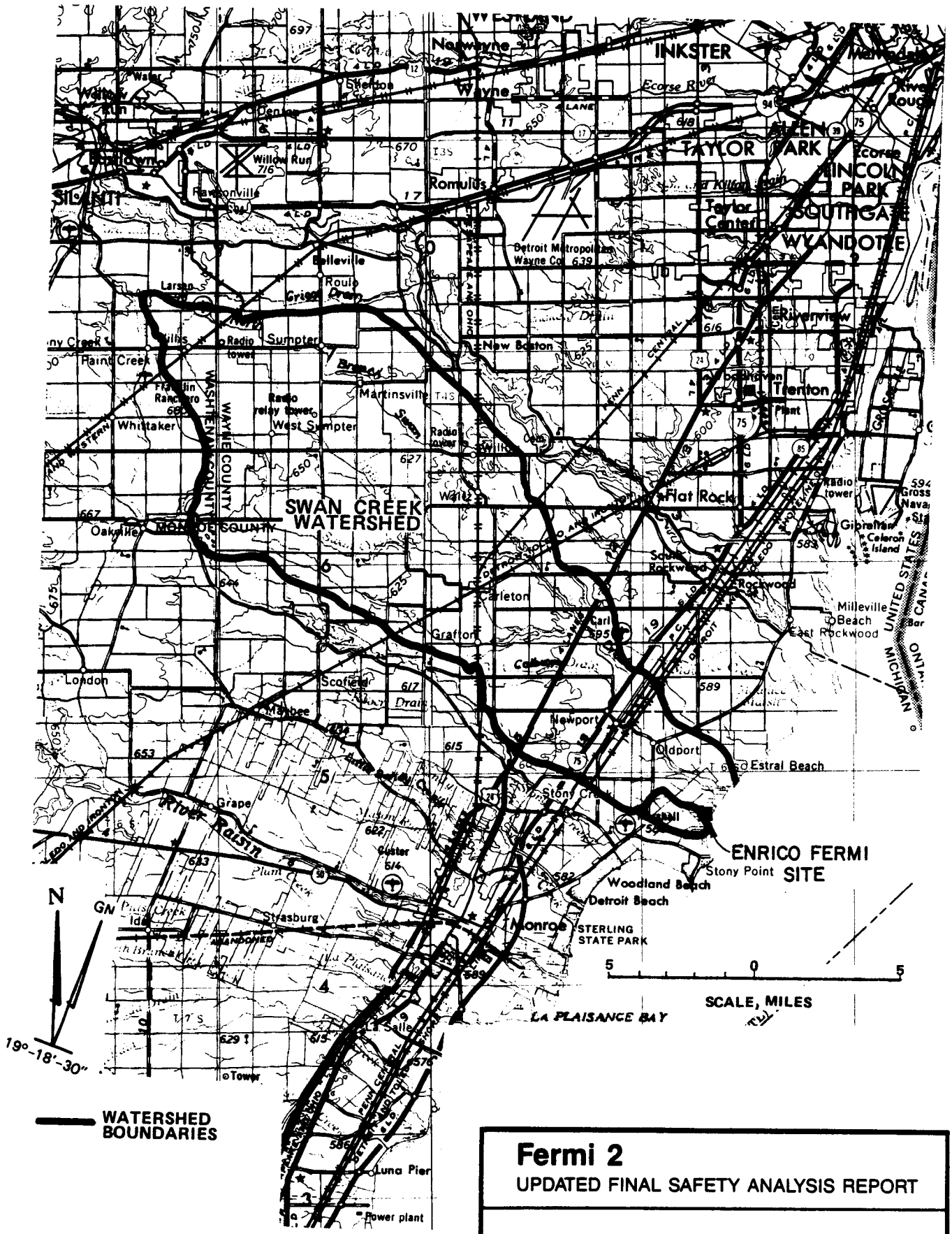
FIGURE 2.4-3

SITE TOPOGRAPHIC MAP



REFERENCE:
PORTIONS OF DETROIT AND TOLEDO
U.S.G.S. TOPOGRAPHIC MAPS.

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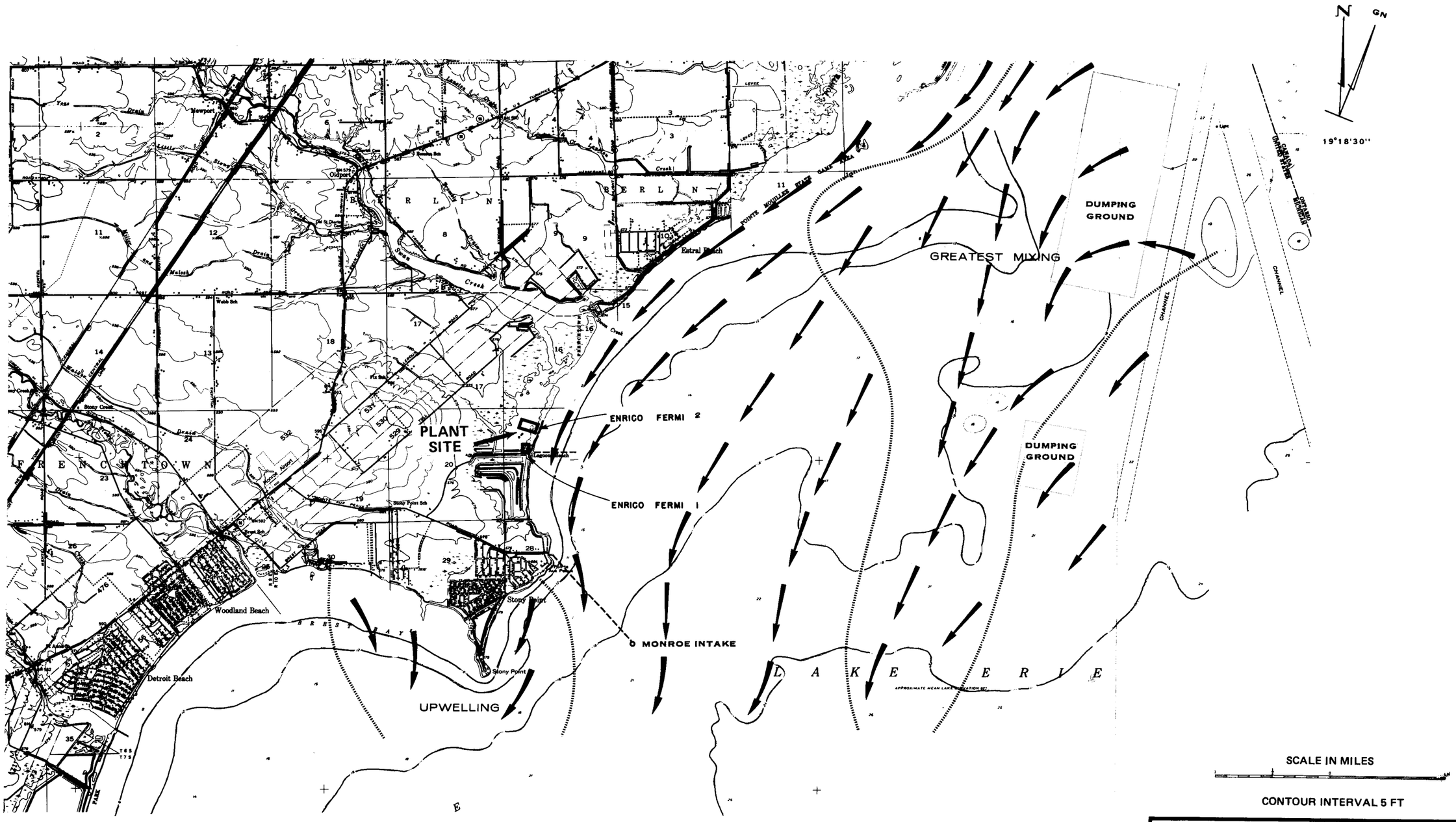
FIGURE 2.4-4

SWAN CREEK WATERSHED



REFERENCE:
THIS MAP WAS PREPARED FROM PORTIONS OF THE FOLLOWING U.S.G.S. TOPOGRAPHIC QUADRANGLES: ESTRAL BEACH, MICHIGAN, 1942, STONY POINT, MICHIGAN, 1952, ROCKWOOD, MICHIGAN, 1952, AND FLAT ROCK, MICHIGAN, 1952.

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FIGURE 2.4-5

WATER CURRENT PATTERNS WITH WINDS FROM NORTHWEST THROUGH NORTHEAST

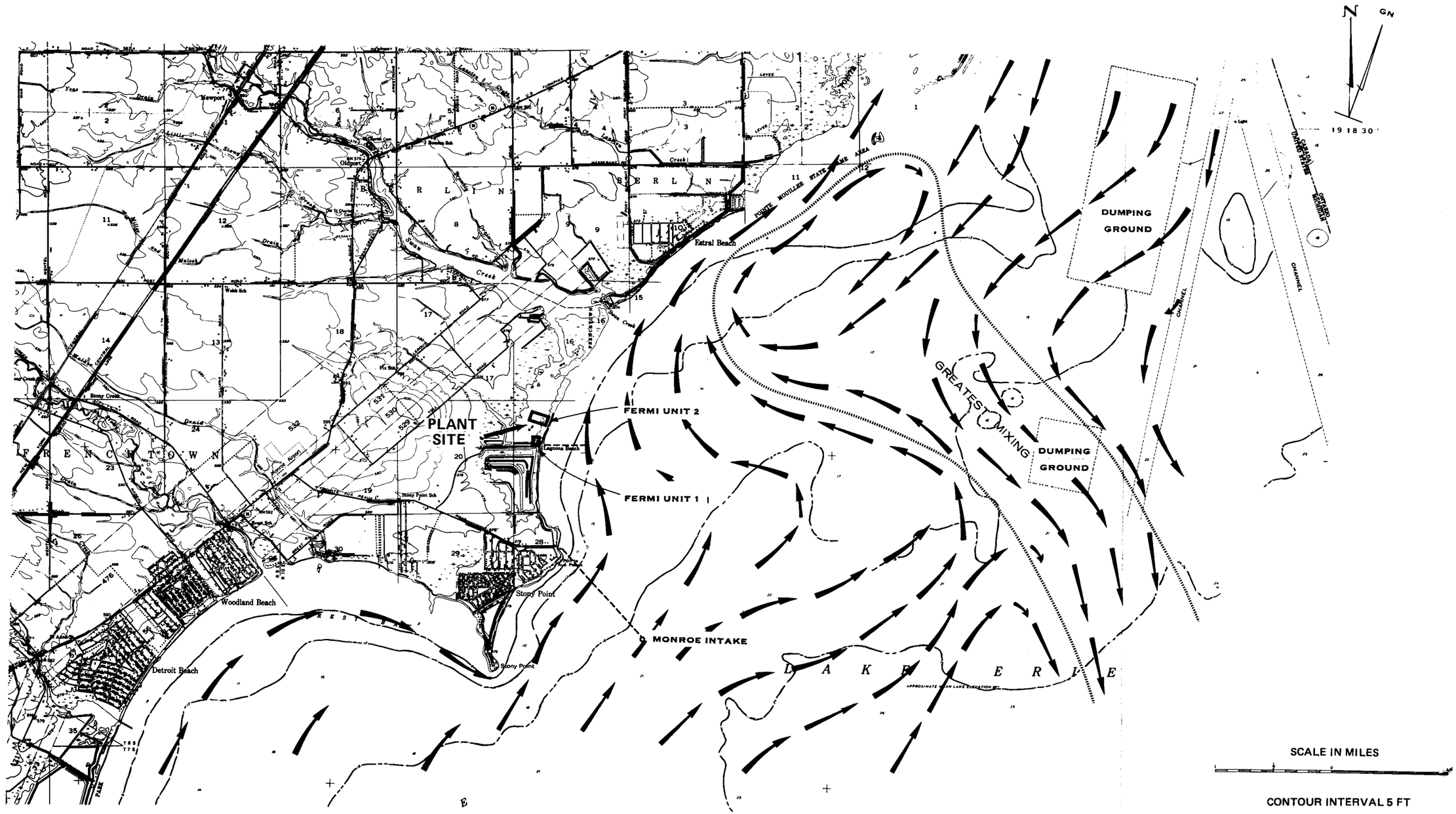

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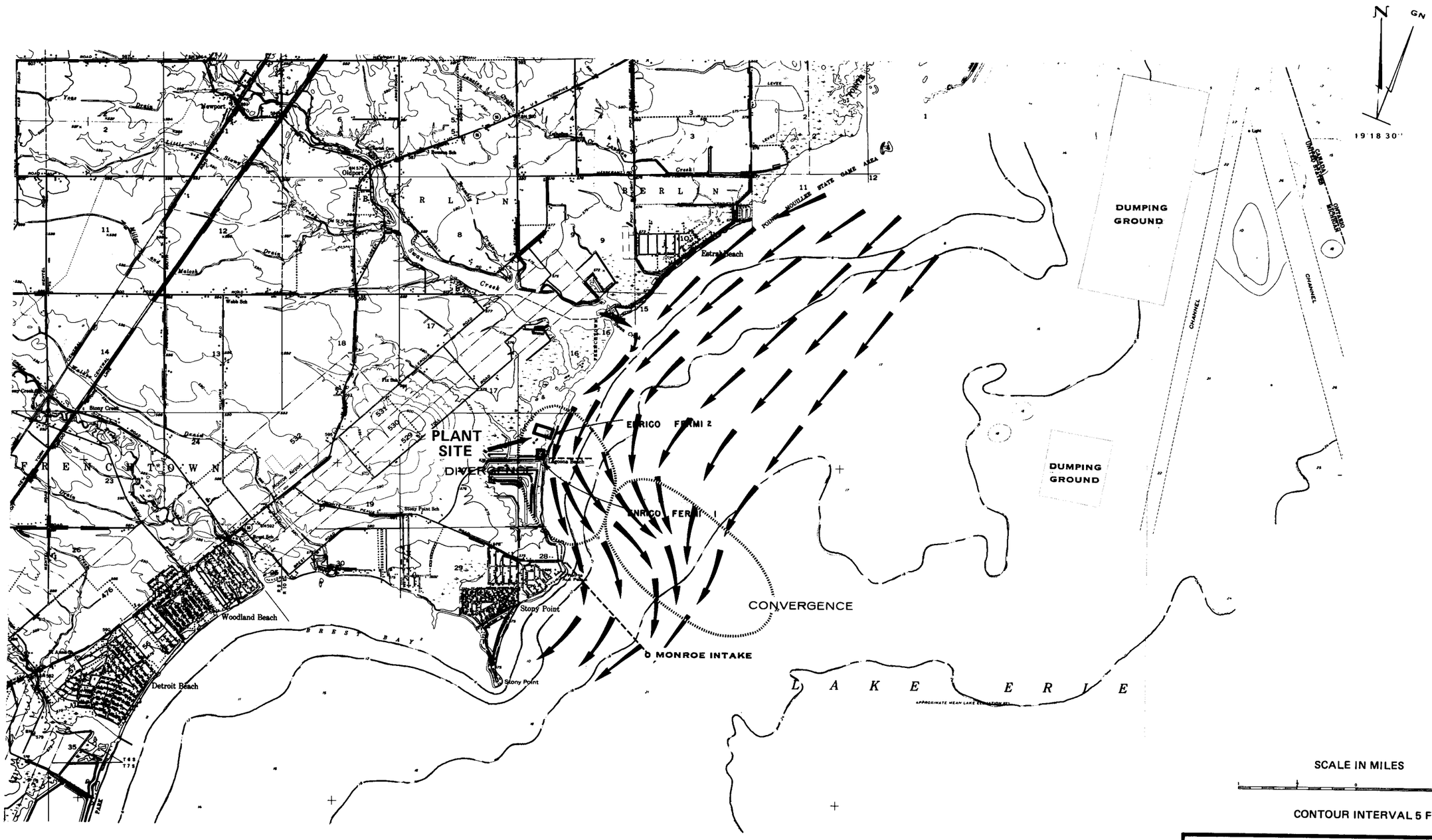
FIGURE 2.4-6

WATER CURRENT PATTERNS WITH WINDS FROM EAST-SOUTHEAST THROUGH WEST

REFERENCE:
THIS MAP WAS PREPARED FROM PORTIONS OF THE FOLLOWING U.S.G.S. TOPOGRAPHIC QUADRANGLES: ESTRAL BEACH, MICHIGAN, 1942, STONY POINT, MICHIGAN, 1952, ROCKWOOD, MICHIGAN, 1952, AND FLAT ROCK, MICHIGAN, 1952.



REFERENCE:
THIS MAP WAS PREPARED FROM PORTIONS OF THE FOLLOWING U.S.G.S. TOPOGRAPHIC QUADRANGLES: ESTRAL BEACH, MICHIGAN, 1942, STONY POINT, MICHIGAN, 1952, ROCKWOOD, MICHIGAN, 1952, AND FLAT ROCK, MICHIGAN, 1952.

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FIGURE 2.4-7

WATER CURRENT PATTERNS UNDER ICE CONDITIONS

FIGURE 2.4-8 HAS BEEN DELETED

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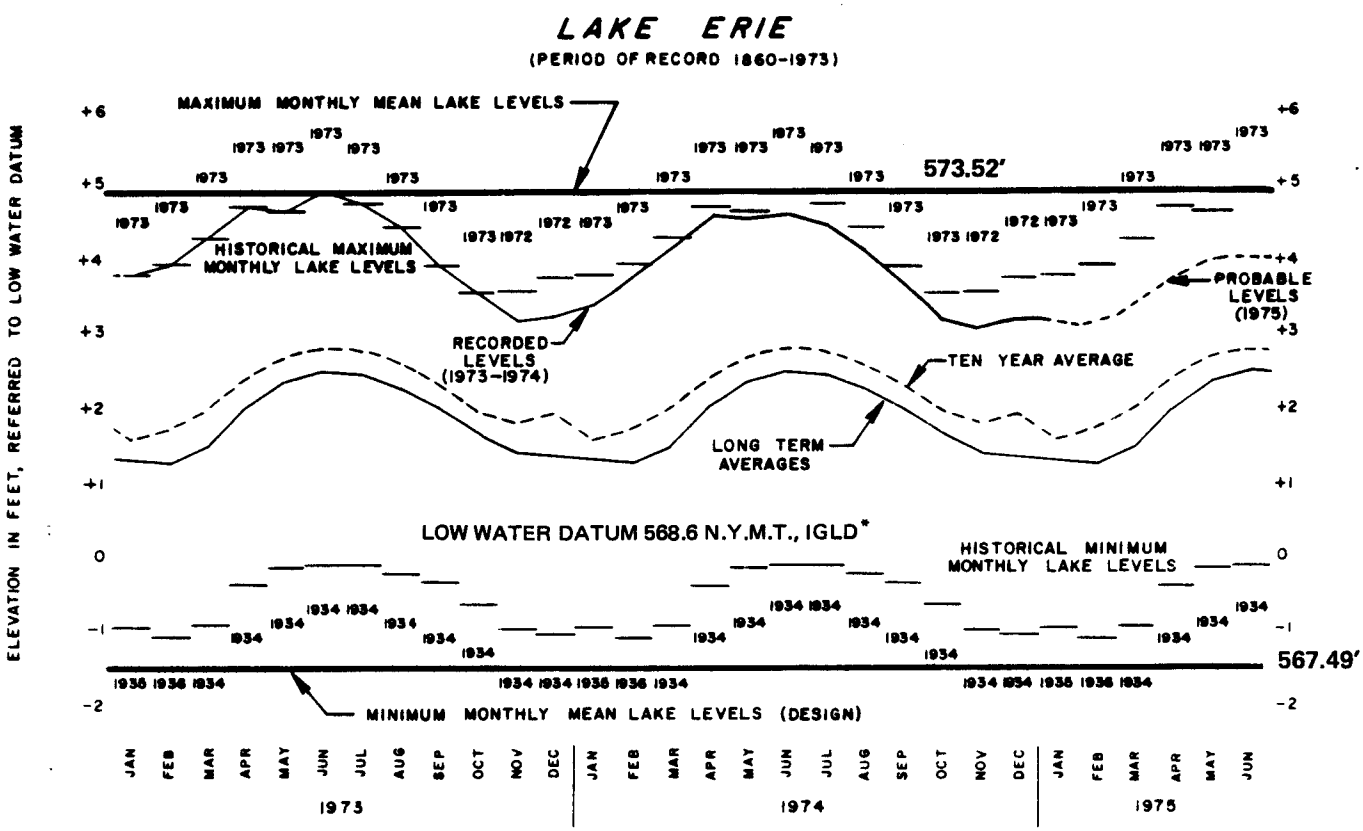

*ADD 1.94' TO CONVERT TO N.Y.M.T., 1935

REFERENCE:
U.S. DEPARTMENT OF COMMERCE, MONTHLY BULLETIN OF LAKE LEVELS FOR JANUARY 1974, NATIONAL OCEAN SURVEY, LAKE SURVEY CENTER.

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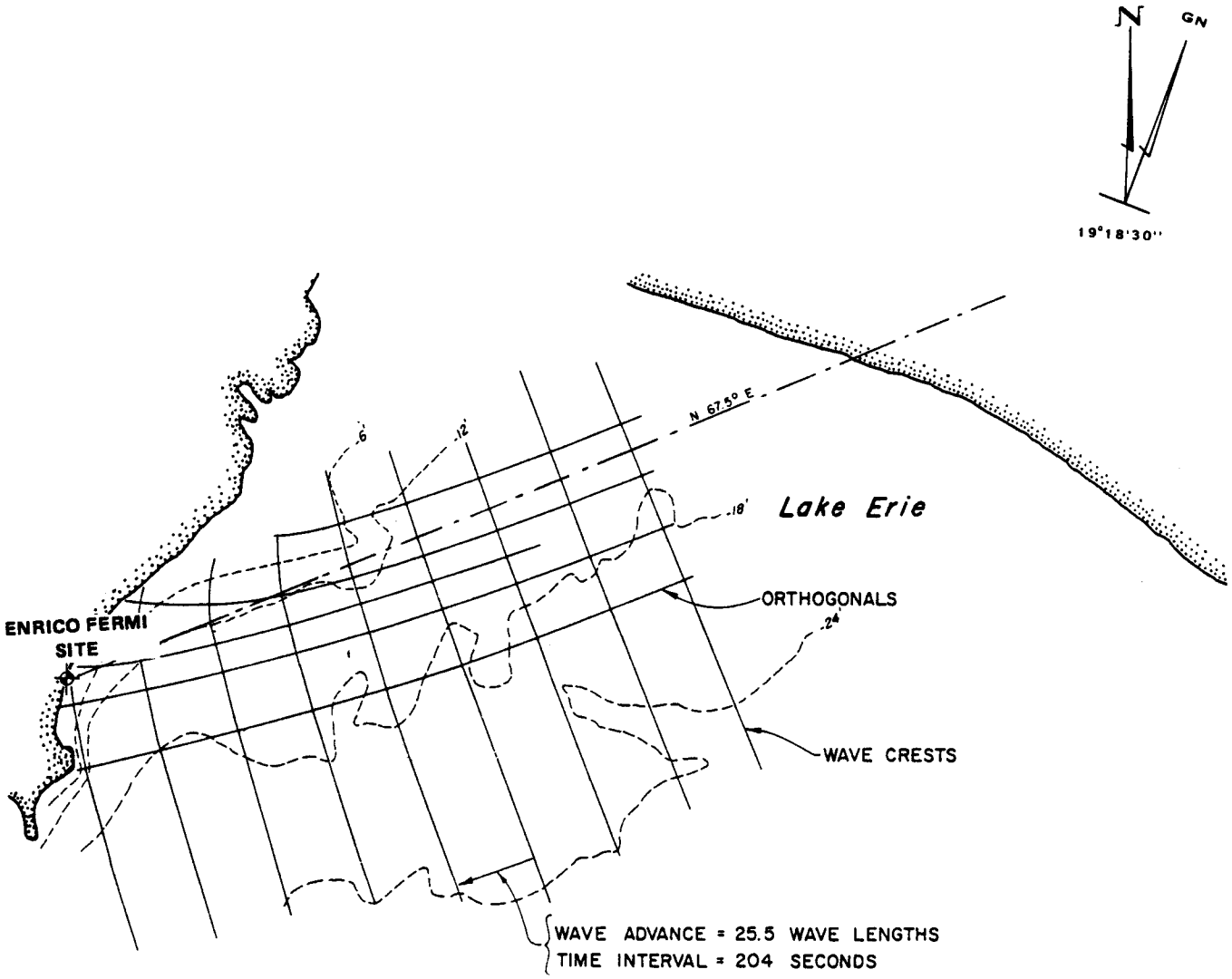
FIGURE 2.4-9

MAXIMUM AND MINIMUM MONTHLY MEAN LAKE LEVELS



LEGEND:

- - - - LAKE BOTTOM CONTOURS
SOUNDING DATUM: NYMT 1935
WAVES REFRACTED DURING TIDE = +16.4 FEET NYMT 1935
WAVE PERIOD = 8.0 SECONDS
WAVE DIRECTION FROM N67.5°E

100,000 50,000 0 100,000

SCALE IN FEET

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FIGURE 2.4-10

WAVE REFRACTION

REFERENCE:
U.S. LAKE SURVEY, CHART NO. 39, 1968

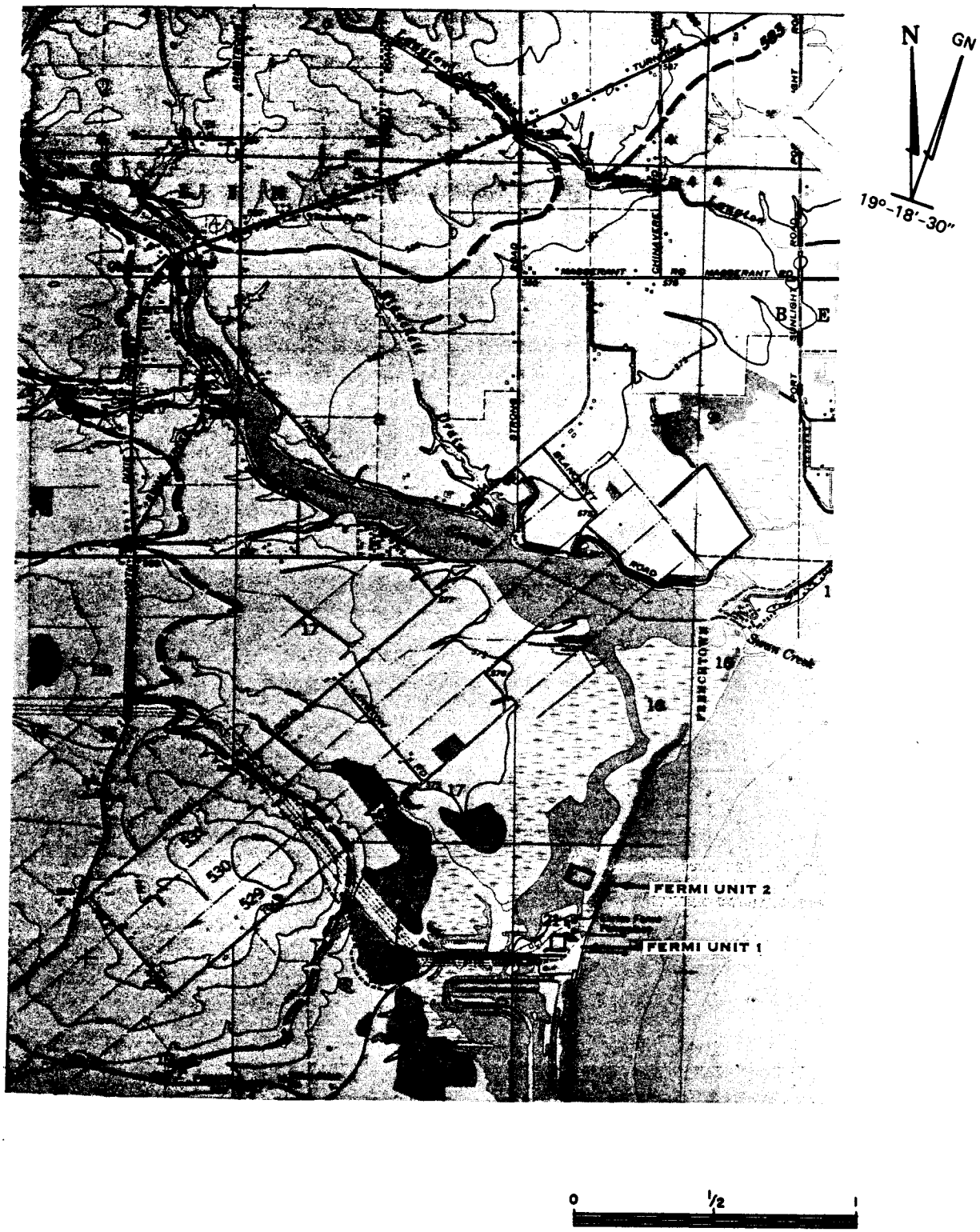

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FIGURE 2.4-11

SITE AREA TOPOGRAPHY SHOWING 583-FT CONTOUR

REFERENCE:
U.S.G.S. TOPOGRAPHIC QUADRANGLE
STONY POINT, MICHIGAN – 1967.

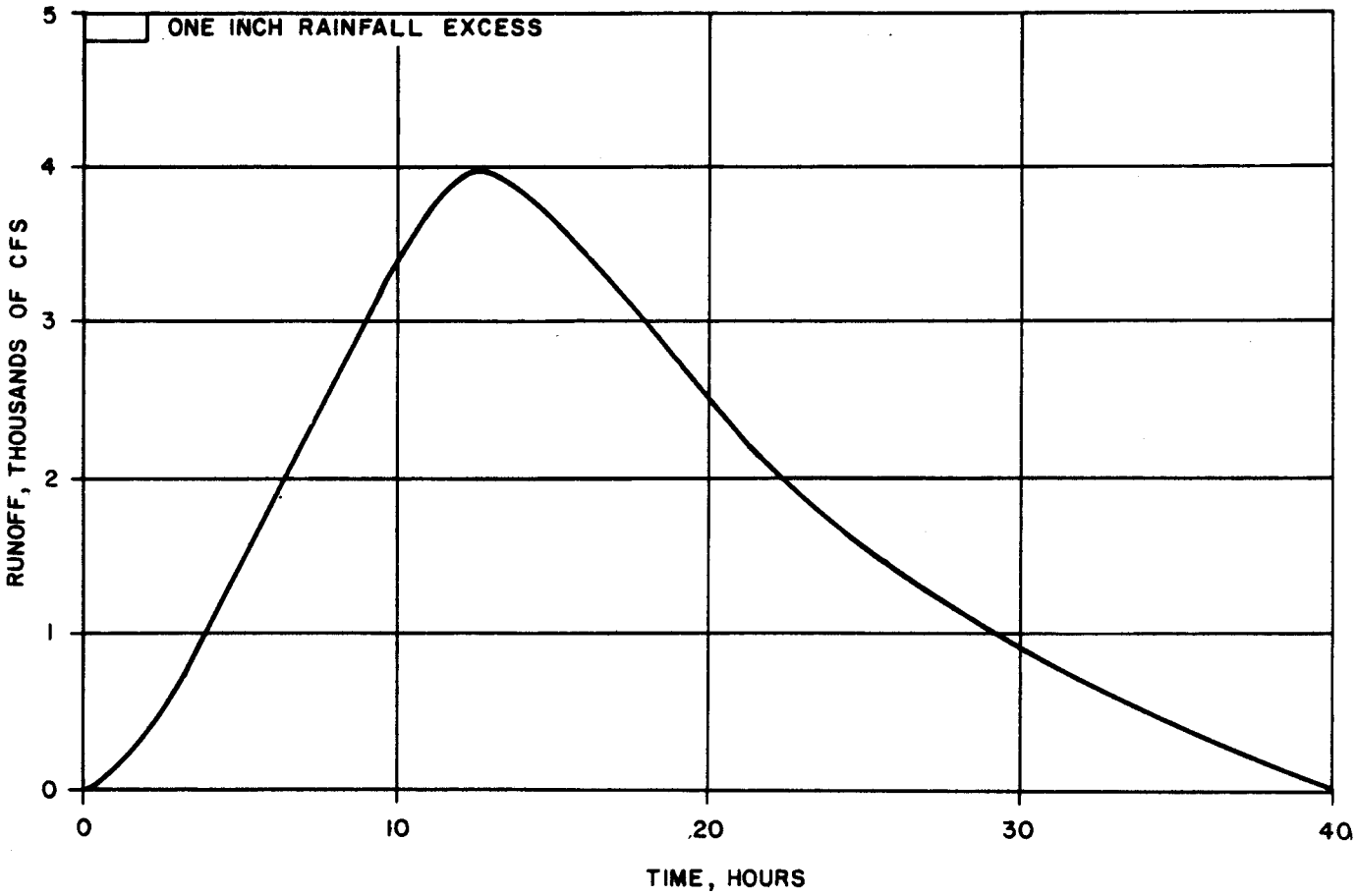

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FIGURE 2.4-12

UNIT HYDROGRAPH – SWAN CREEK AT MOUTH

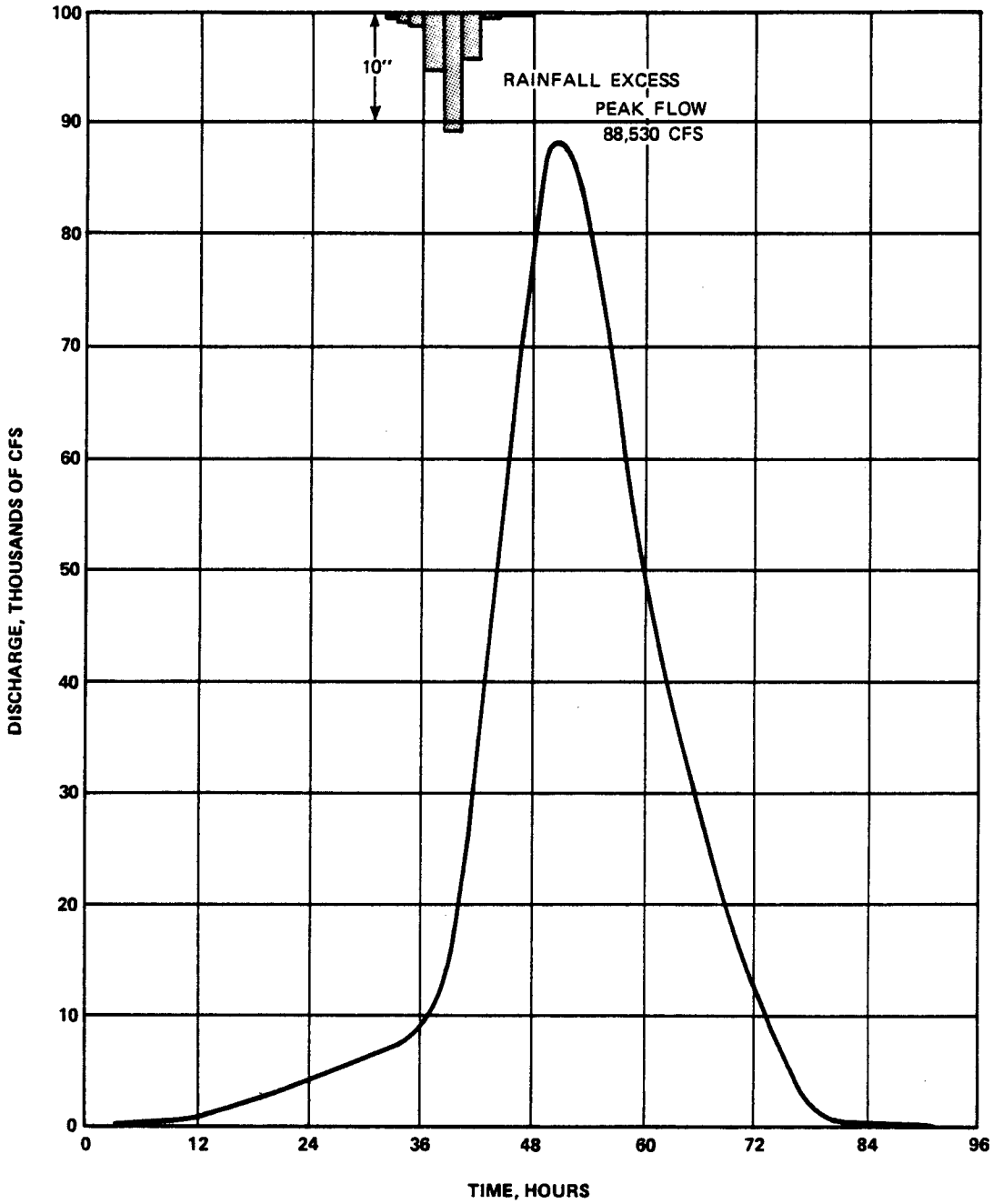

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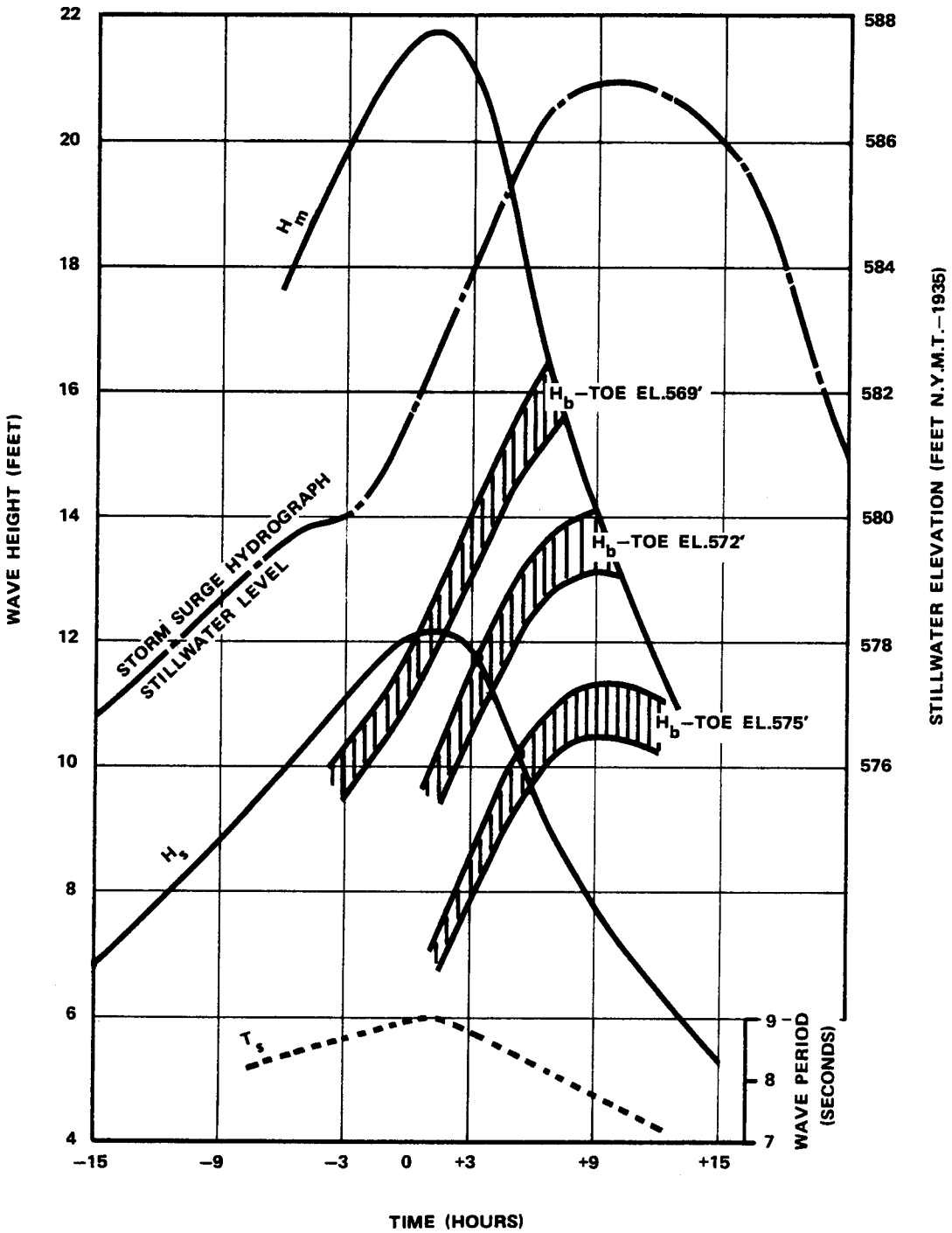
FIGURE 2.4-13

PMF HYDROGRAPH – SWAN CREEK AT MOUTH



LEGEND:

H_m = MAXIMUM HEIGHT

H_s = SIGNIFICANT WAVE HEIGHT

H_b = BREAKING WAVE HEIGHT FOR SHORE BARRIER TOR ELEVATION (UPPER LIMIT CONSIDERS WAVE SETUP)

T_s = SIGNIFICANT WAVE PERIOD

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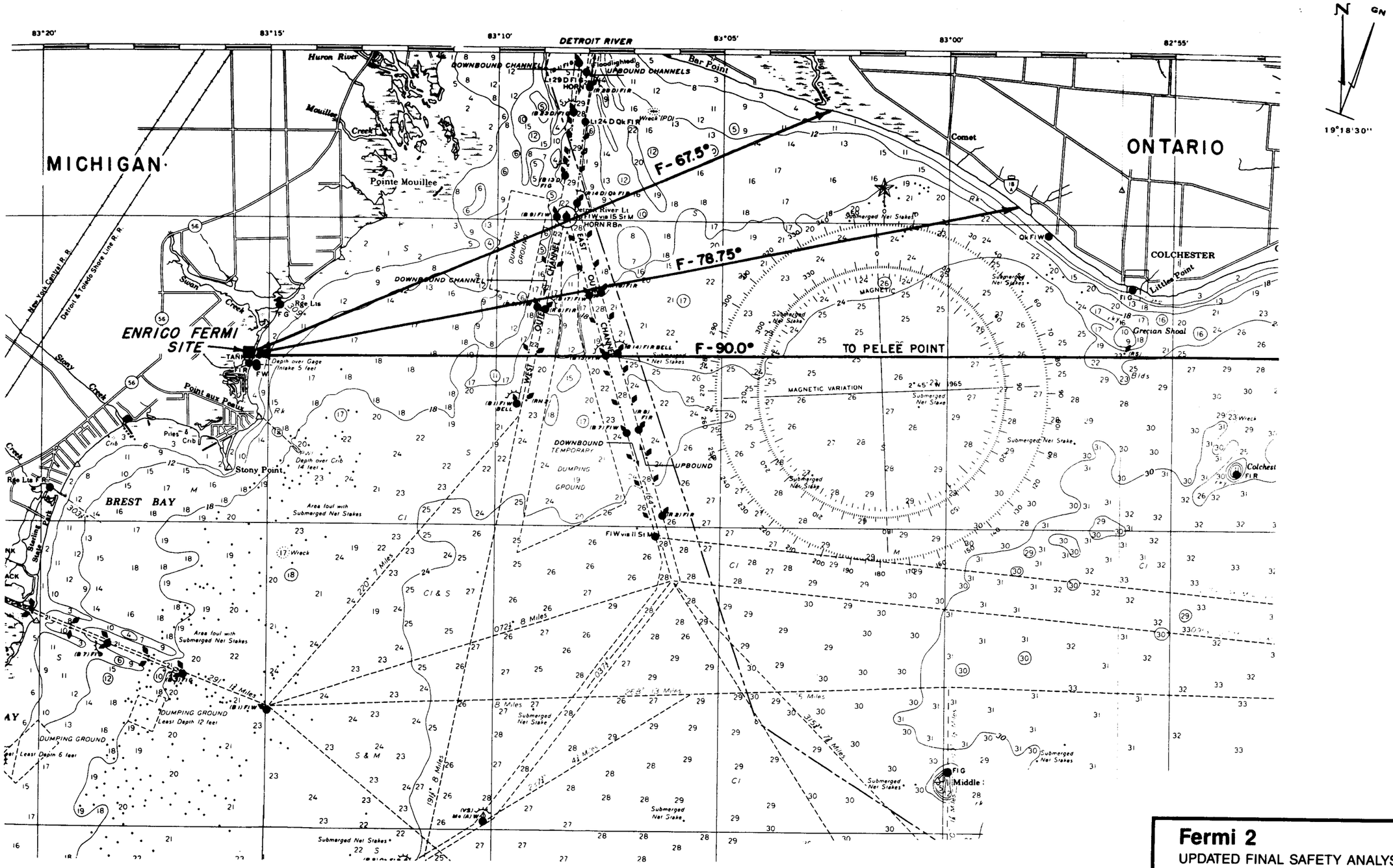
FIGURE 2.4-14

STORM SURGE HYDROGRAPH FOR PMME



REFERENCE:
U.S. LAKE SURVEY, CHART NO. 39, 1968

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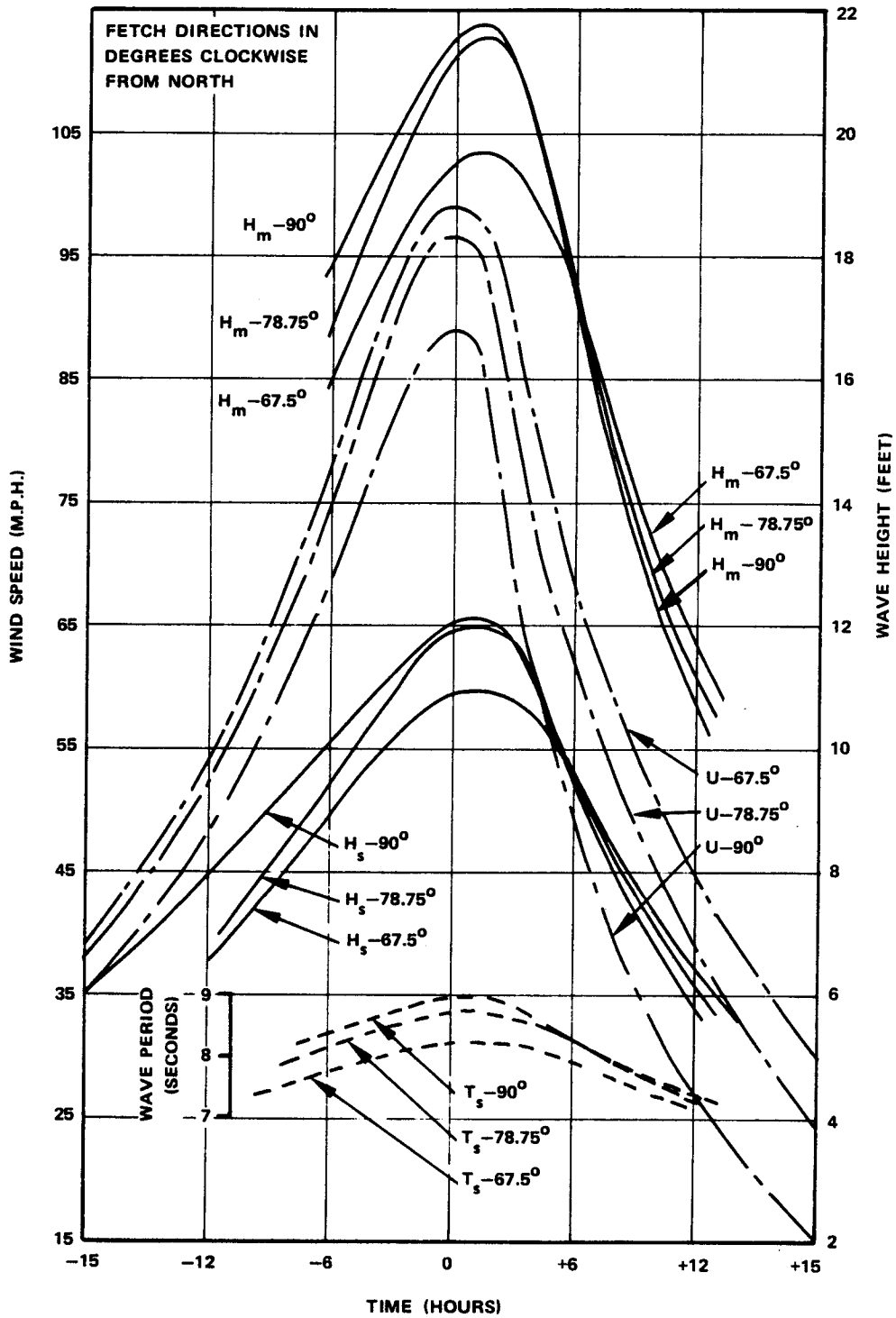
FIGURE 2.4-15

FETCH DIRECTIONS



LEGEND:

H_m = MAXIMUM HEIGHT

H_s = SIGNIFICANT WAVE HEIGHT

T_s = SIGNIFICANT WAVE PERIOD

U = COMPONENT WIND VELOCITY

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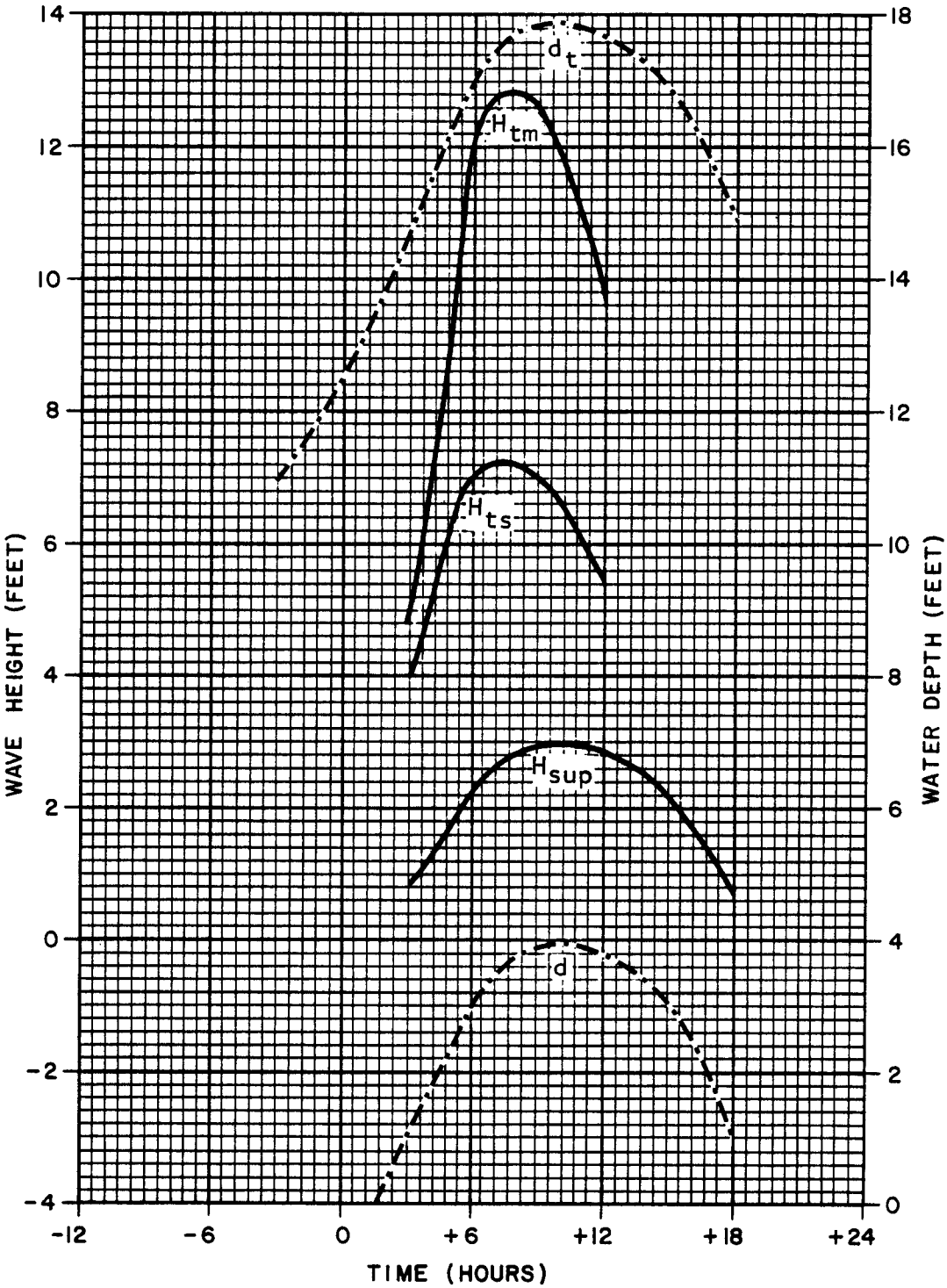
FIGURE 2.4-16

WIND AND WAVE CHARACTERISTICS VERSUS TIME



LEGEND:
ALL ELEVATIONS REFER TO NYMT, 1935.

FOR A SHORE BARRIER TOE ELEVATION OF 569.0 FT AND CREST ELEVATION OF 583.0 FT:

H_{tm} = WAVE HEIGHT TRANSMITTED OVER SHORE BARRIER FOR INCIDENT MAXIMUM WAVE HEIGHTS

H_{ts} = WAVE HEIGHT TRANSMITTED OVER SHORE BARRIER FOR INCIDENT SIGNIFICANT WAVE HEIGHTS

H_{sup} = MAXIMUM WAVE HEIGHT SUPPORTED OVER INLAND FLOODED PLANT GRADE (ELEVATION 583.0 FT) WITHOUT BREAKING

d_t = DEPTH OF WATER AT SHORE BARRIER WITH A TOE ELEVATION OF 569.0 FT

d = INLAND DEPTH OF WATER ABOVE PLANT GRADE ELEVATION OF 583.0 FT.

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FIGURE 2.4-17

TRANSMITTED AND SUPPORTED WAVE HEIGHTS VERSUS TIME

STATIC FORCES		BREAKING WAVE (MINIKIN METHOD)			NON-BREAKING WAVE (1) (SAINFLOU METHOD)			BROKEN WAVE
PRESSURE (PSF)		2,960			2,925			3,060
THRUST (LBS./FT. OF WALL)		70,100			68,700			75,000
DYNAMIC FORCES	WAVE PERIOD (SECONDS)	3.4	7.7	9.0	3.4	7.7	9.0	FORCES ARE INDEPENDENT OF WAVE PERIOD
PRESSURE (PSF)	10% SLOPE	2,460	660	520	150	180	182	122
	5% SLOPE	3,000	900	700				
THRUST (LBS./FT. OF WALL)	10% SLOPE	2,460	660	520	1,125	1,235	1,245	256
	5% SLOPE	3,000	900	700				

CASE 1

D = 46.9' (DEPTH FROM STILLWATER LEVEL TO TOP OF REACTOR SLAB)
d = 3.9' (DEPTH FROM STILLWATER LEVEL TO TOP OF PLANT GRADE)
H = 3.0' (WAVE HEIGHT)

STATIC FORCES		BREAKING WAVE (MINIKIN METHOD)			NON-BREAKING WAVE (1) SAINFLOU METHOD)			BROKEN WAVE
PRESSURE (PSF)		3,100			2,925			3,160
THRUST (LBS./FT. OF WALL)		77,000			68,700			80,100
DYNAMIC FORCES	WAVE PERIOD (SECONDS)	4.5	7.7	9.0	4.5	7.7	9.0	FORCES ARE INDEPENDENT OF WAVE PERIOD
PRESSURE (PSF)	10% SLOPE	4,480	1,870	1,460	268	312	319	215
	5% SLOPE	5,500	2,460	1,950				
THRUST (LBS./FT. OF WALL)	10% SLOPE	8,060	3,360	2,640	3,664	3,900	3,950	814
	5% SLOPE	9,900	4,430	3,520				

(1) DYNAMIC FORCES OF NON-BREAKING WAVES RESULT FROM CLAPOTIS AFFECT.

CASE 2

D = 46.9' (DEPTH FROM STILLWATER LEVEL TO TOP OF REACTOR SLAB)
d = 6.9' (DEPTH FROM STILLWATER LEVEL TO TOP OF PLANT GRADE)
H = 5.4' (WAVE HEIGHT)

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FIGURE 2.4-18

WAVE PRESSURE AND FORCES AGAINST REACTOR BUILDING

STATIC FORCES		BREAKING WAVE (MINIKIN METHOD)			NON-BREAKING WAVE (1) (SAINFLOU METHOD)			BROKEN WAVE
PRESSURE (PSF)		2,334			2,240			2,371
THRUST (LBS./FT. OF WALL)		43,641			40,208			45,049
DYNAMIC FORCES	WAVE PERIOD (SECONDS)	3.4	7.7	9.0	3.4	7.7	9.0	FORCES ARE INDEPENDENT OF WAVE PERIOD
PRESSURE (PSF)	10% SLOPE	2,460	660	520	150	180	182	122
	5% SLOPE	3,000	900	700				
THRUST (LBS./FT. OF WALL)	10% SLOPE	2,460	660	520	1,125	1,235	1,245	256
	5% SLOPE	3,000	900	700				

CASE 1

D = 35.9' (DEPTH FROM STILLWATER LEVEL TO TOP OF RHR SLAB)
d = 3.9' (DEPTH FROM STILLWATER LEVEL TO TOP OF PLANT GRADE)
H = 3.0' (WAVE HEIGHT)

STATIC FORCES		BREAKING WAVE (MINIKIN MMETHOD)			NON-BREAKING WAVE (1) (SAINFLOU METHOD)			BROKEN WAVE
PRESSURE (PSF)		2,409			2,240			2,477
THRUST (LBS./FT. OF WALL)		46,487			40,208			49,174
DYNAMIC FORCES	WAVE PERIOD (SECONDS)	4.5	7.7	9.0	4.5	7.7	9.0	FORCES ARE INDEPENDENT OF WAVE PERIOD
PRESSURE (PSF)	10% SLOPE	4,480	1,870	1,460	268	312	319	215
	5% SLOPE	5,500	2,460	1,950				
THRUST (LBS./FT. OF WALL)	10% SLOPE	8,060	3,360	2,640	3,664	3,900	3,950	814
	5% SLOPE	9,900	4,430	3,520				

(1) DYNAMIC FORCES OF NON-BREAKING WAVES RESULT FROM CLAPOTIS AFFECT.

CASE 2

D = 35.9' (DEPTH FROM STILLWATER LEVEL TO TOP OF RHR SLAB)
d = 6.9' (DEPTH FROM STILLWATER LEVEL TO TOP OF PLANT GRADE)
H = 5.4' (WAVE HEIGHT)

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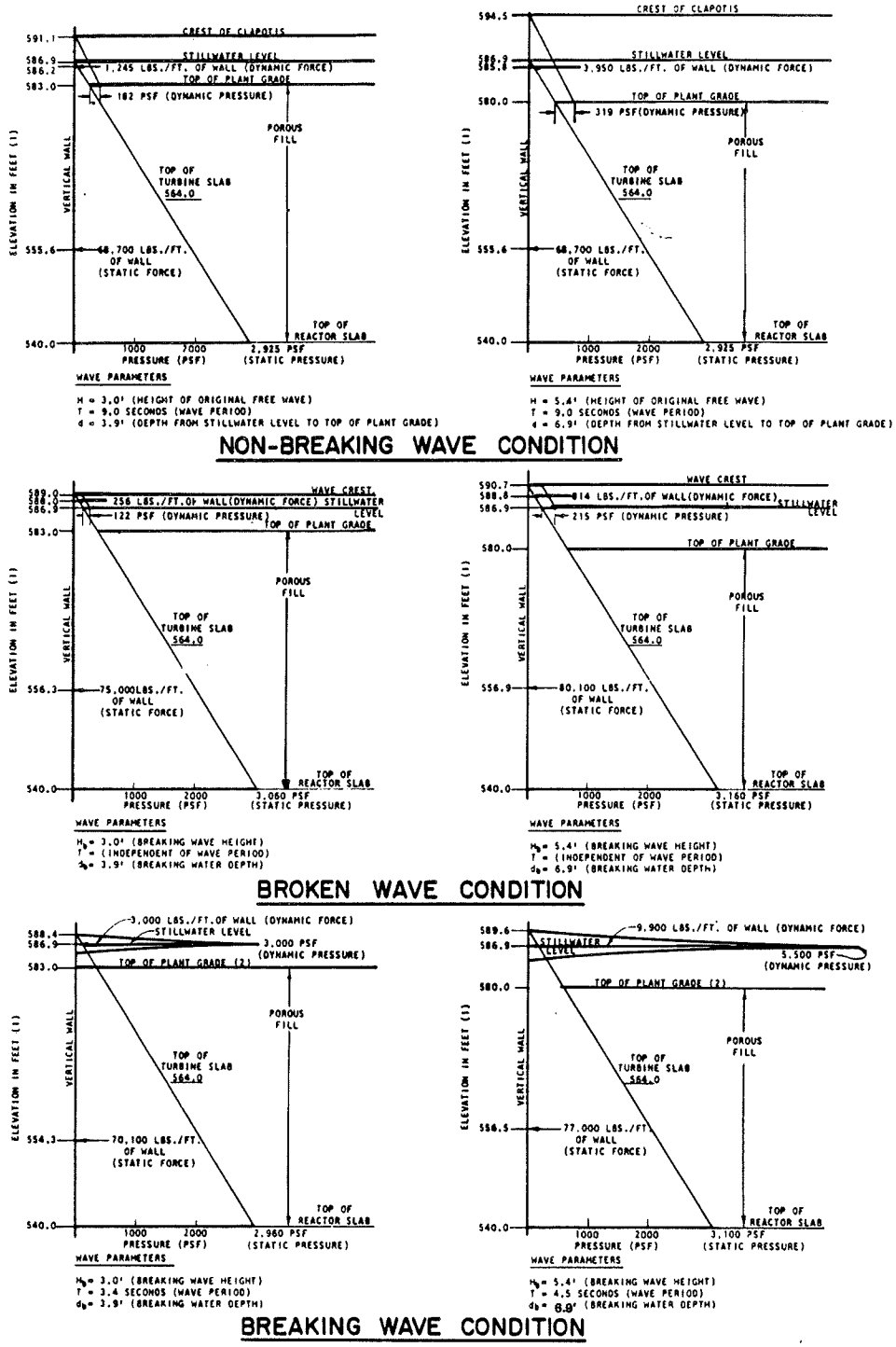
FIGURE 2.4-19

WAVE PRESSURE AND FORCES AGAINST RESIDUAL HEAT REMOVAL COMPLEX



NOTES:

1. **ALL ELEVATIONS REFER TO NYMT, 1935 DATUM**
2. **5 PERCENT SLOPE ASSUMED**

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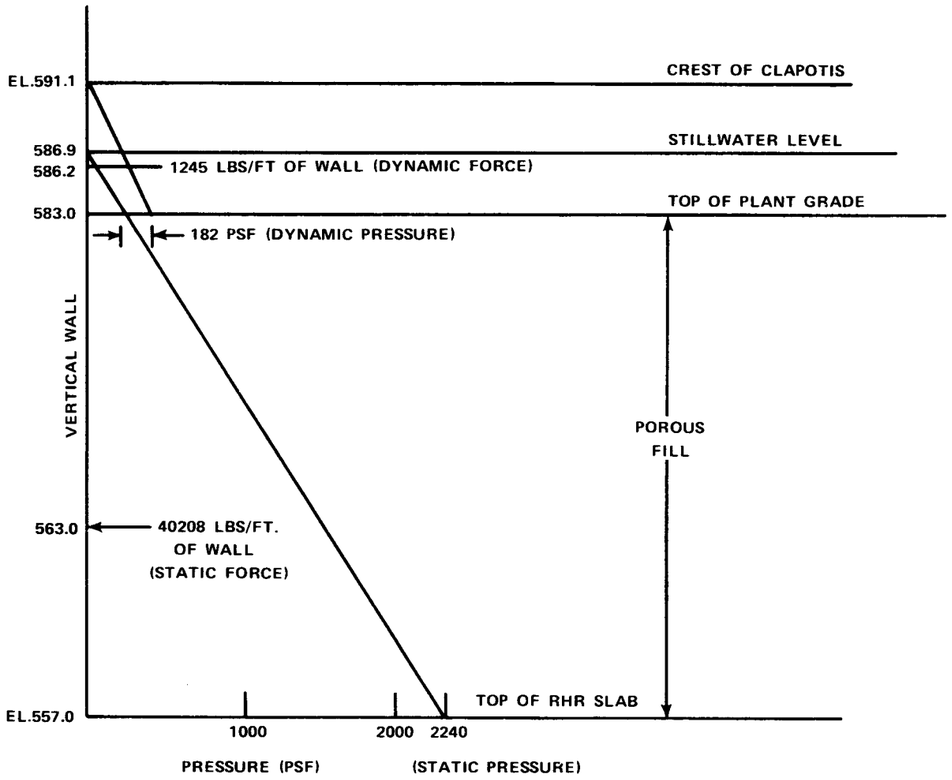
FIGURE 2.4-20

WAVE PRESSURE DISTRIBUTIONS AGAINST REACTOR/AUXILIARY BUILDING

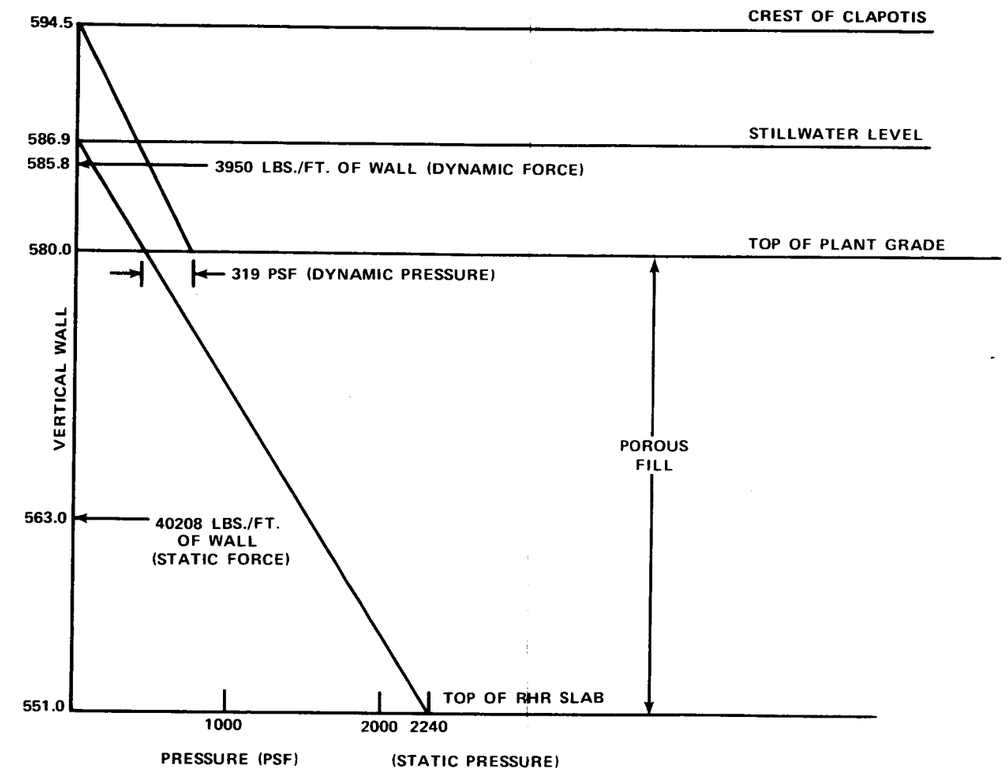


WAVE PARAMETERS

H = 3.0′ (HEIGHT OF ORIGINAL FREE WAVE)

T = 9.0 (SECONDS (WAVE PERIOD)

d = 3.9′ (DEPTH FROM STILLWATER LEVEL TO TOP OF PLANT GRADE)



WAVE PARAMETERS

H = 5.4′ (HEIGHT OF ORIGINAL FREE WAVE)

T = 9.0 SECONDS (WAVE PERIOD)

d = 6.9′ (DEPTH FROM STILLWATER LEVEL TO TOP OF PLANT GRADE)

NON–BREAKING WAVE CONDITION

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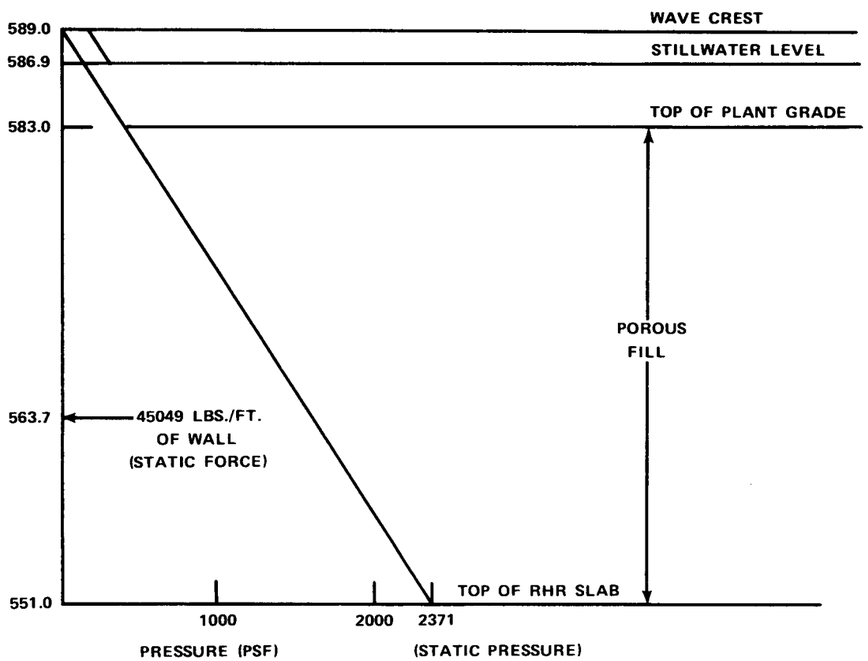
FIGURE 2.4-21, SHEET 1

WAVE PRESSURE DISTRIBUTION AGAINST RESIDUAL HEAT REMOVAL COMPLEX

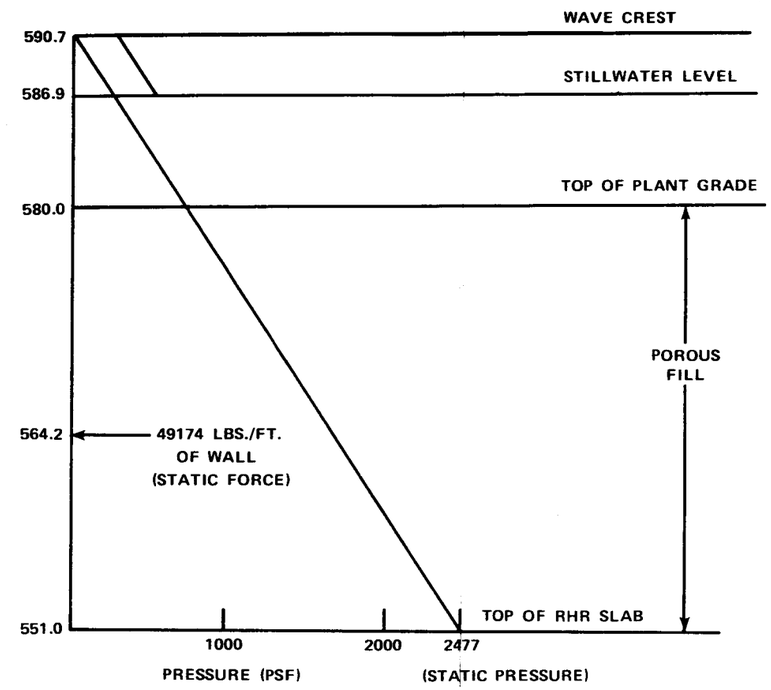


WAVE PARAMETERS

H_b = 3.0′ (BREAKING WAVE HEIGHT)

T = (INDEPENDENT OF WAVE PERIOD)

d_b = 3.9′ (BREAKING WATER DEPTH)



WAVE PARAMETERS

H_b = 5.4 (BREAKING WAVE HEIGHT)

T = (INDEPENDENT OF WAVE PERIOD)

d_b = 6.9′ (BREAKING WATER DEPTH)

BROKEN WAVE CONDITION

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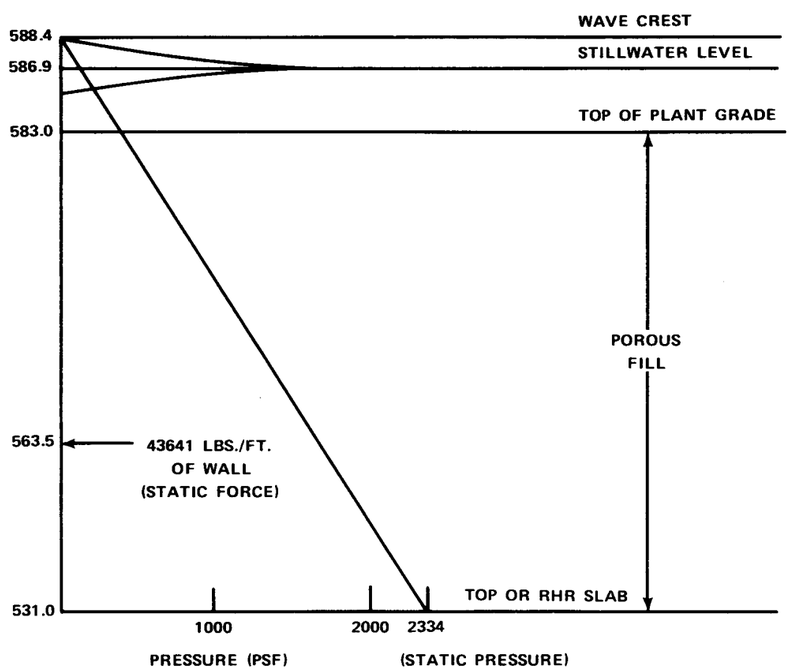
FIGURE 2.4-21, SHEET 2

WAVE PRESSURE DISTRIBUTION AGAINST RESIDUAL HEAT REMOVAL COMPLEX

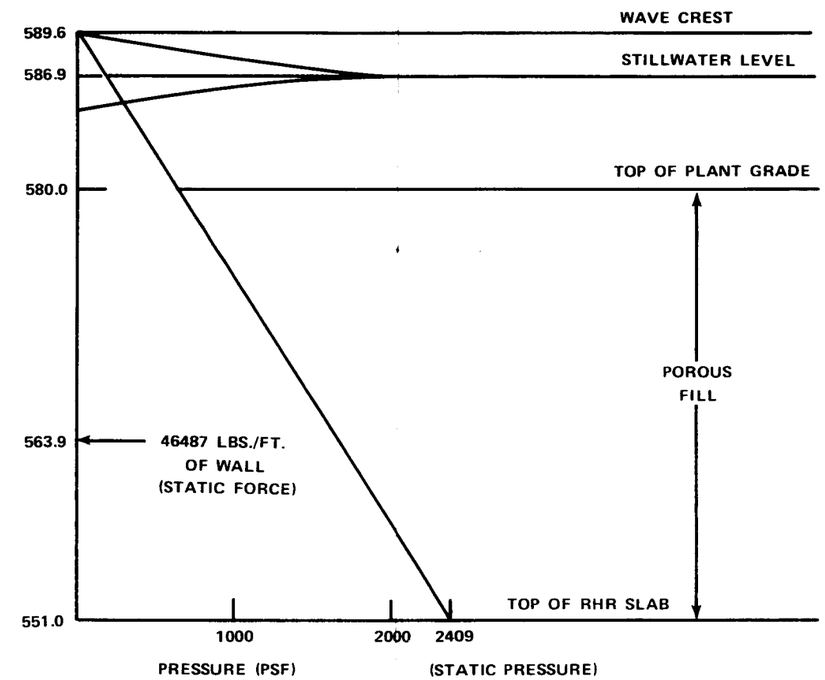


WAVE PARAMETERS

H_b = 3.0′ (BREAKING WAVE HEIGHT)

T = 3.4 SECONDS (WAVE PERIOD)

d_b = 3.9′ (BREAKING WATER DEPTH)



WAVE PARAMETERS

H_b = 5.4′ (BREAKING WAVE HEIGHT)

T = 4.5 SECONDS (WAVE PERIOD)

d_b = 6.9 (BREAKING WATER DEPTH)

BREAKING WAVE CONDITION

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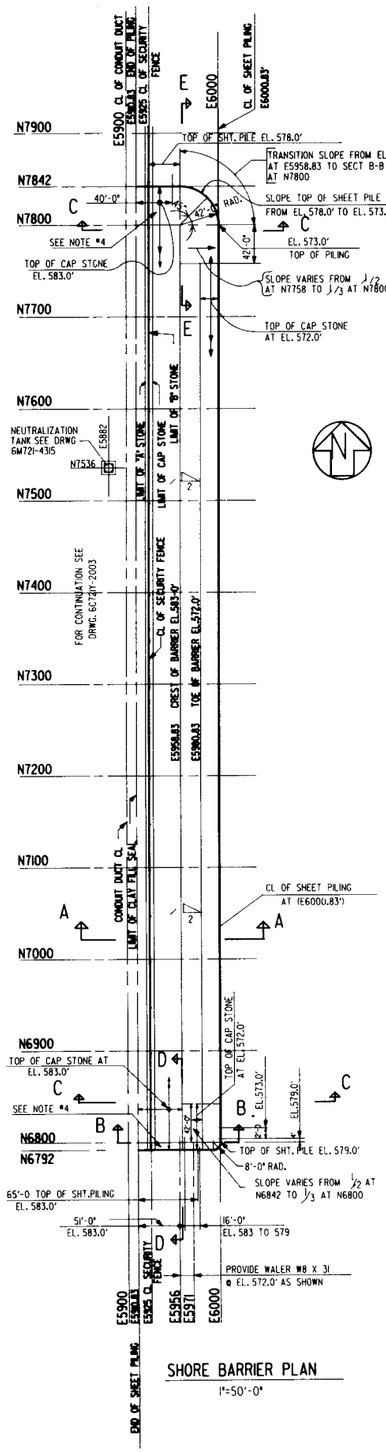
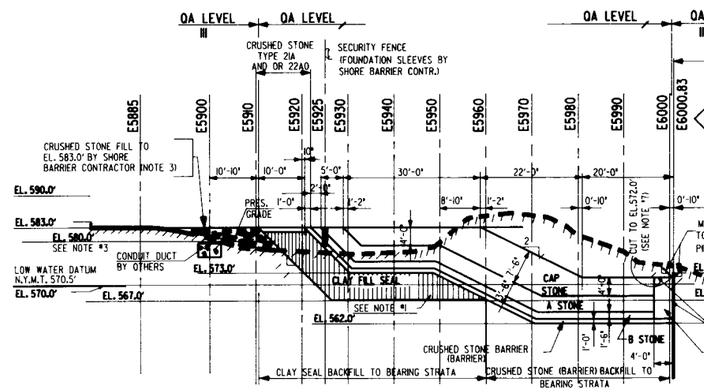
FIGURE 2.4-21, SHEET 3

WAVE PRESSURE DISTRIBUTION AGAINST RESIDUAL HEAT REMOVAL COMPLEX



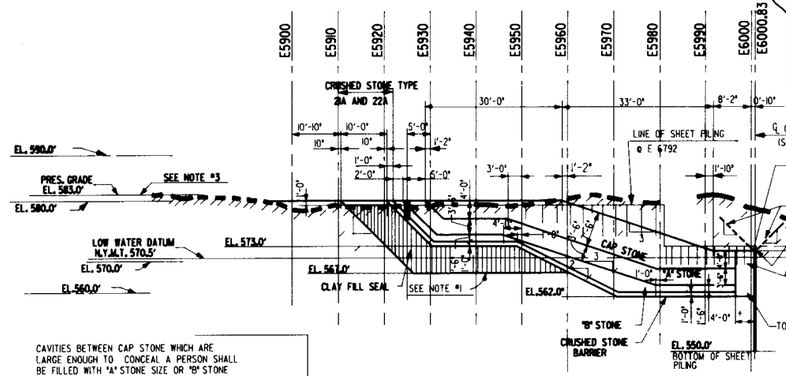
SHORE BARRIER PLAN

1"=50'-0"



SECTION A-A

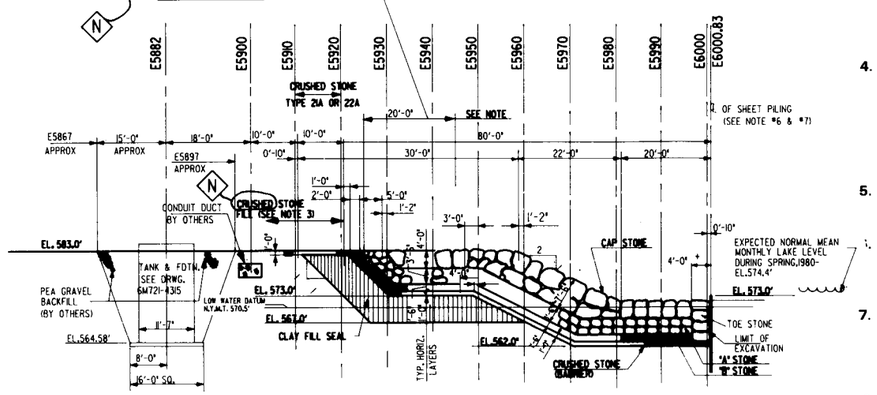
1"=10'-0"



SECTION B-B

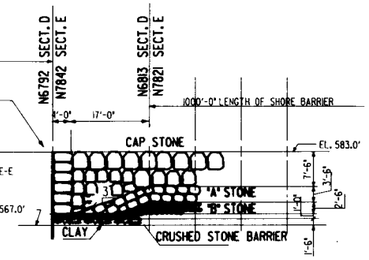
1"=10'-0"

CAVITIES BETWEEN CAP STONE WHICH ARE LARGE ENOUGH TO CONCEAL A PERSON SHALL BE FILLED WITH "A" STONE SIZE OR "B" STONE SIZE MATERIAL. CAVITY FILL AREAS ARE IN A ZONE FROM THE SECURITY FENCE (E-5925) TO E-5945 AND SHALL BE DETERMINED BY A FIELD SURVEY OF THE COMPLETED SHORE BARRIER. FILL MATERIAL IS CLASSIFIED AS Q.A. LEVEL III.



SECTION C-C

TYPICAL SHORE BARRIER SECTION (1"=10'-0")



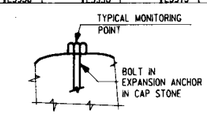
SECTION D-D (AS SHOWN)
SECTION E-E (AS NOTED)

TYPICAL SHORE BARRIER END

1"=10'-0"

CAP STONE MONITORING POINTS

STA. COORD.	AS BUILT ELEV.	STA. COORD.	AS BUILT ELEV.	STA. COORD.	AS BUILT ELEV.	STA. COORD.	AS BUILT ELEV.	STA. COORD.	AS BUILT ELEV.	STA. COORD.	AS BUILT ELEV.	STA. COORD.	AS BUILT ELEV.
N6809 E5816	580.05	N6901 E5904	581.52	N7119 E5947	582.09	N7228 E5966	578.59	N7429 E5957	581.40	N7601 E5937	583.85	N7722 E5971	576.42
N6824 E5941	581.63	N6898 E5958	579.89	N7122 E5967	581.45	N7233 E5971	575.59	N7429 E5963	578.12	N7610 E5920	582.21	N7814 E5949	581.86
N6809 E5957	576.99	N6905 E5977	577.08	N7120 E5964	578.72	N7328 E5946	582.22	N7428 E5974	576.53	N7618 E5961	582.56	N7809 E5925	581.11
N6825 E5956	581.01	N7020 E5949	580.92	N7121 E5974	575.52	N7322 E5965	581.18	N7529 E5948	583.04	N7616 E5977	576.58	N7814 E5965	578.88
N6826 E5968	579.02	N7023 E5960	580.59	N7222 E5951	582.54	N7337 E5966	578.99	N7531 E5961	582.10	N7721 E5940	583.15	N7815 E5975	577.81
N6809 E5972	575.10	N7023 E5967	578.58	N7223 E5950	582.70	N7328 E5974	575.09	N7531 E5965	579.91	N7721 E5956	582.08		
N6822 E5976	577.65	N7024 E5974	575.99	N7225 E5958	581.22	N7422 E5965	582.16	N7526 E5973	575.13	N7718 E5963	579.82		

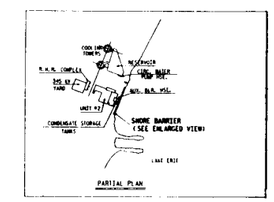


NOTES:

1. EXCAVATE PEAT AND OTHER UNSUITABLE MATERIAL TO EXPOSE ACCEPTABLE BEARING STRATA PER SPEC. 3071-176. BORINGS INDICATE THIS STRATA STARTS AT APPROX. 567'.
2. BACKFILL WITH CLAY FILL SEAL OR CRUSHED STONE (BARRIER) PER SECTION A-A.
3. FILL AREAS BETWEEN BARRIER AND EXISTING SITE FILL WITH CRUSHED STONE TO EL. 583.0 FT. A SMOOTH TRANSITION TO THE EXISTING PLANT GRADE IS REQUIRED. CRUSHED STONE SHALL CONFORM TO MDOT STANDARD SPEC. FOR CONSTRUCTION CLASS 21 A. PLACED AT 120 PCF IN 12" LAYERS.
4. REAR SEGMENT OF SHORE BARRIER END SHALL BE FILLED WITH A COMPOSITE OF CAP STONE, "A" STONE, AND "B" STONE WITH THE HEAVIER STONES ON OUTER LAYERS. THE MATERIAL SHALL BE PLACED IN A MANNER TO PROVIDE A SMOOTH TRANSITION BETWEEN THE HORIZONTAL GRADE AT THE REAR OF THE BARRIER AND THE 3 ON 1 SLOPE ON THE FLANK.
5. EXPECTED NORMAL MEANS MONTHLY LAKE LEVEL AND LOW WATER DATUM ARE IN REFERENCE TO NEW YORK MEAN TIDE (N.Y.M.T., 1935).
6. SHEET PILING SHALL BE PZ27 (UNITED STATES STEEL DESIGNATION) OR EQUAL WITH STEEL BRAND OR GRADE OF ASTM-328 AND MINIMUM YIELD POINT OF 38500 PSI.
7. DRIVE SHEET PILING TO BEDROCK OR A MAXIMUM DEPTH OF EL. 550.0' PRIOR TO EXCAVATION WITH TOP AT EL. 575.0'+ AND CUT TO EL. 572.0' AFTER ARMOR UNITS HAVE BEEN PLACED. SHEET PILING LENGTH APPROX. 25'-0" ±.
8. THIS WORK IS GOVERNED BY 3071-176 PRIOR TO EXCAVATION, THE GROUND EITHER SIDE OF THE SHEETING MUST BE EXCAVATED TO ELEV. 572'-0".
9. SHORE BARRIER QA LEVEL SHALL BE AS NOTED ON SECTION A-A.
10. SHEETING MATERIAL & INSTALLATION SHALL BE QA LEVEL III. SEE SPEC. 3071-176 FOR QA REQUIREMENTS.

LEGEND:

CAP STONE & TOE STONE:	2 LAYERS TOTAL 7.5' THICK AND 4.0' THICK RESP. MIN. SIZE - 3.0 TONS MAX. SIZE - 5.0 TONS 50% BY WEIGHT SHALL CONSIST OF PIECES WEIGHING OVER 4 TONS
"A" STONE:	2 LAYERS, TOTAL 3.5' THICK LESS THAN 10% BY WEIGHT UNDER 500 POUNDS LESS THAN 50% BY WEIGHT UNDER 800 POUNDS LESS THAN 10% BY WEIGHT OVER 2000 POUNDS
"B" STONE:	2 LAYERS TOTAL 1.5' THICK MAX. SIZE - 200 POUNDS NOT LESS THAN 10% NOR MORE THAN 35% BY WEIGHT CONSISTING OF PIECES WEIGHING LESS THAN 20 POUNDS
CRUSHED STONE: (BARRIER)	MINIMUM 1.0' THICK MAX. DIAMETER - 6 INCHES NOT LESS THAN 10% NOR MORE THAN 35% BY WEIGHT CONSISTING OF FINES WITH DIAMETERS LESS THAN 3 INCHES
ALTERNATE:	CAP STONE, TOE STONE AND "A" STONE SUBSTITUTED BY CONCRETE BLOCKS - 4000 PSI TYPE V CEMENT AIR ENTRAINED CONCRETE (SEE SPEC. 3071-176)



NOTE*

VARIATIONS BETWEEN THE AS BUILT ELEVATIONS AND DESIGN ELEVATIONS OF CAP STONE EXCEED THE SPECIFIED TOLERANCE OF +6 INCHES FOR MANY AREAS OF THE SHORE BARRIER. THESE VARIATIONS HAVE BEEN EVALUATED BY THE SHORE BARRIER ENGINEER AND ARE ACCEPTABLE AS REPORTED IN DAMES & MOORE LETTER DMO-01 DATED JULY 16, 1984 AND SARGENT & LUNDY LETTER SLS 2668.

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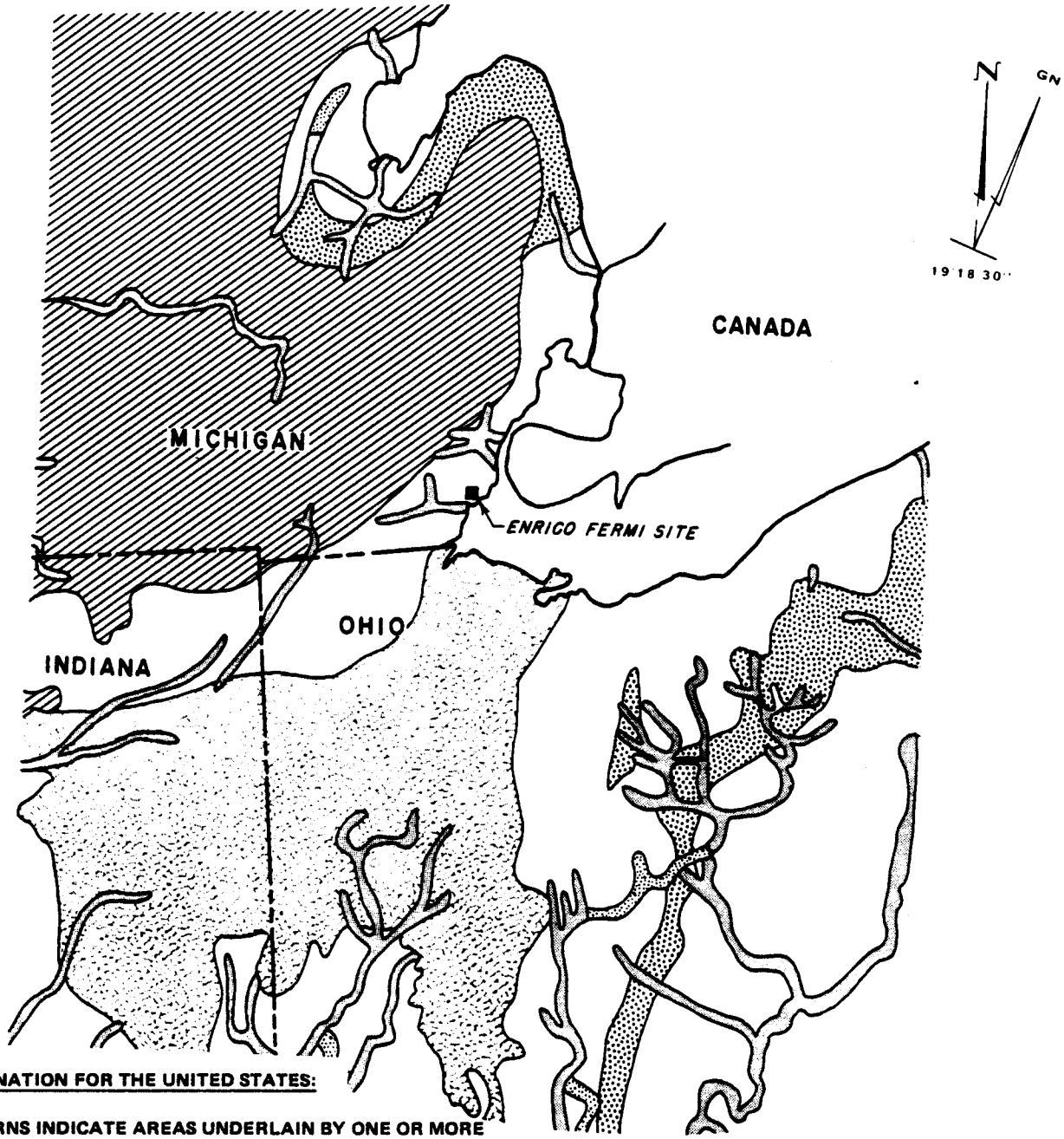
FIGURE 2.4-22

SHORE BARRIER DESIGN

DETROIT EDISON COMPANY DRAWING NO. 6C721-0040, REV. N



EXPLANATION FOR THE UNITED STATES:

PATTERNS INDICATE AREAS UNDERLAIN BY ONE OR MORE AQUIFERS GENERALLY CAPABLE OF YEILDING TO A WELL AT LEAST 50 gpm OF WATER CONTAINING NOT MOR THAN 2000 ppm OF DISSOLVED SOLIDS (INCLUDING AREAS WHERE MORE HIGHLY MINERALIZED WATER IS ACTUALLY USED).

LEGEND:

UNCONSOLIDATED AND SEMICONSOLIDATED AQUIFERS

- ALLUVIAL SAND AND GRAVEL
- WATERCOURSE – ALLUVIAL VALLEY TRAVERSED BY PERENNIAL STREAM FROM WHICH RECHARGE CAN BE INDUCED
- SURFICIAL ALLUVIAL VALLEY NO LONGER TRAVERSED BY PERENNIAL STREAM (ABANDONED WATERCOURSE), OR BURIED ALLUVIAL VALLEY

CONSOLIDATED – ROCK AQUIFERS

- SANDSTONE (INCLUDES SOME SAND)
- CARBONATE ROCKS (LIMESTONE AND DOLOMITE; LOCALLY INCLUDE GYPSUM)

50 0 50

SCALE, MILES

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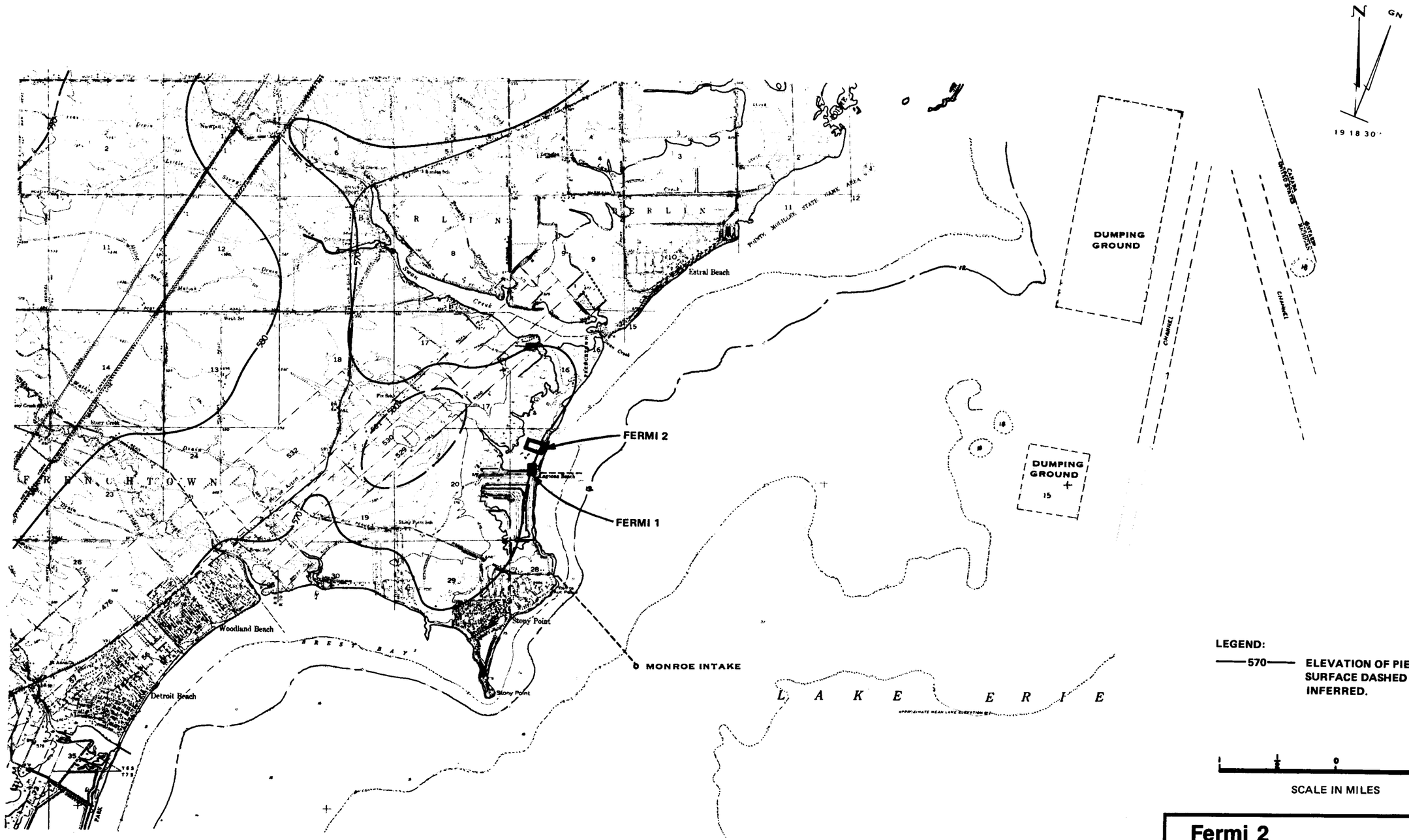
FIGURE 2.4-23

REGIONAL AQUIFER DISTRIBUTION

REFERENCE:
U.S. DEPARTMENT OF THE INTERIOR GEOLOGICAL SURVEY WATER SUPPLY PAPER NO. 1800, 1963.



REFERENCE:
THIS MAP WAS PREPARED FROM PORTIONS OF THE FOLLOWING U.S.G.S. TOPOGRAPHIC QUADRANGLES: ESTRAL BEACH, MICHIGAN, 1942, STONY POINT, MICHIGAN, 1952, ROCKWOOD, MICHIGAN, 1952, AND FLAT ROCK, MICHIGAN, 1952.

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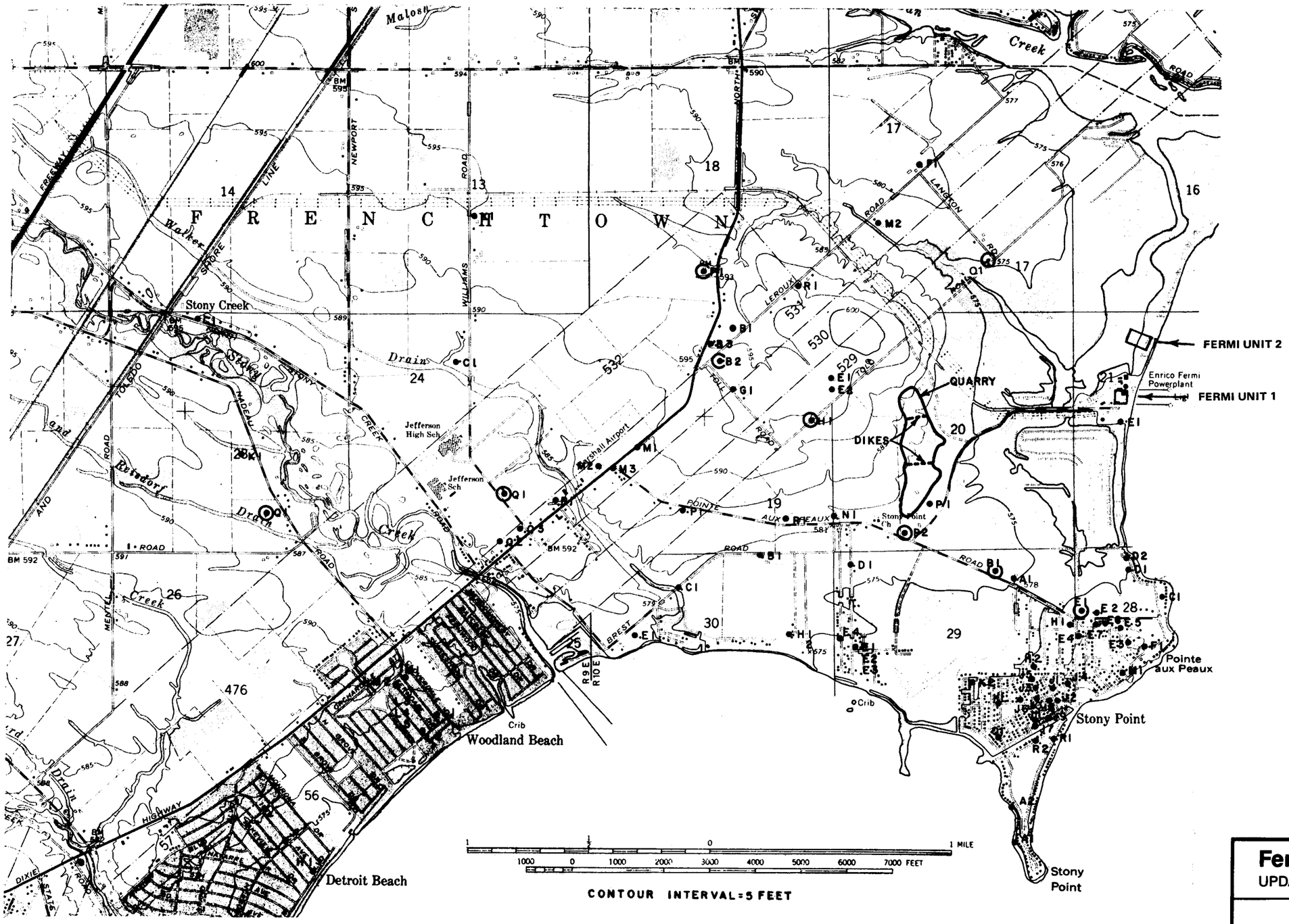
FIGURE 2.4-24

PIEZOMETRIC SURFACE 1961-1966



LEGEND:

- ● WELL LOCATION (SEE TABLE 2.4-7 FOR EXPLANATION OF WELL NUMBERING SYSTEM.)
- ⊙ WELL WITH HYDROGRAPH PLOT.

REFERENCE:
U.S.G.S. TOPOGRAPHIC QUADRANGLE
STONY POINT, MICHIGAN – 1967.

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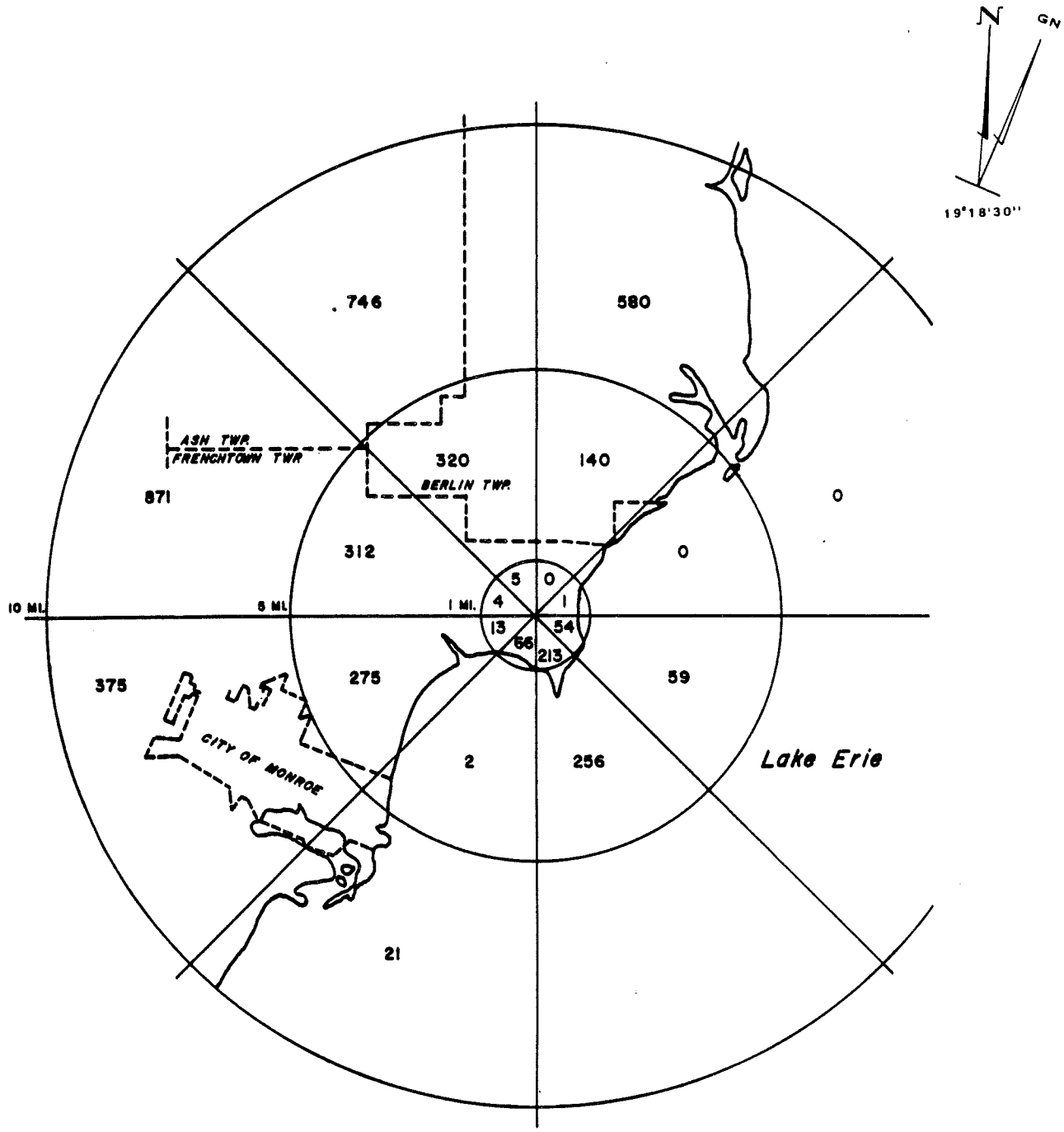
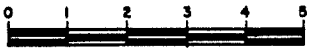
FIGURE 2.4-25

WELL LOCATIONS



LEGEND:
NUMBERS REFER TO NUMBER OF
GROUNDWATER WELLS IN EACH SECTOR.



SCALE IN MILES

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FIGURE 2.4-26

WATER WELL DISTRIBUTION