SECTION 2

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SECTION 2

2.0 SITE CHARACTERISTICS

2.1 GEOGRAPHY AND DEMOGRAPHY

2.1.1 Site Location

The Containment Shield Building for the Davis-Besse Station is located at 41degree°35’49” North latitude and 83degree°05’16” West longitude. The site borders the southwestern shore of Lake Erie and is located in Ottawa County in Northwestern Ohio. The upland in the vicinity of the site is generally agricultural in nature with the topography being very flat. The Universal Transverse Mercator Coordinates are 4607000 N and 326100 E.

Lake Erie, which borders the site for approximately 3,000 ft., is one of the Great Lakes and is important for commerce, commercial and sport fishing, recreation, and water supply to Ohio, and other states which border the lake, and Canada.

Bordering the lake in the vicinity of the site and on the site itself are marshlands which are important to the area for wildlife, particularly waterfowl. To the west of the site is the main unit of the Ottawa National Wildlife Refuge, the State of Ohio Magee Marsh Wildlife Area and Crane Creek State Park. A portion of the station site is leased to the U.S. Government as a National Wildlife Refuge.

A narrow strip of marshland on the southern boundary of the site separates the site from the Toussaint River except for a small segment of the site which extends to the river. This river, which becomes a stream six miles upstream from the mouth, empties into Lake Erie 700 ft. from the lake shoreline site boundary and has a drainage area of approximately 143 square miles to the west of the site.

The location of the station site and geographic features of the surrounding general area are shown on Figure 2.1-1. The Federal and State Wildlife areas are shown on Figure 2.1-2.

2.1.2 Site Description

The site consists of 954 acres of which approximately 733 acres are marshland. The major station structures are located approximately in the center of the site area, 3,000 ft. from the shoreline, and the Containment Building is located 2,400 ft. from the nearest site boundary, which is to the north.

The site, site arrangement, and location of the major station structures are shown on Figure 1.2-12. The site boundary shown on this figure is also the limit of the exclusion area. Ownership of the site area, within the site boundary, resides in the Energy Harbor Nuclear Generation LLC.

The marsh area of the site is surrounded by dikes or upland area which isolate this area from direct communication with adjacent marsh areas, the lake and other waterways. This marsh area is also separated into two separate sections by the intake canal dikes and each is provided with a water level control pump installation to artificially control the marsh water level independent of lake groundwater level.

The marsh and beach ridge areas of the site encompassing 733 acres are leased to the U. S. Government as a National Wildlife Refuge which is administered by the U.S. Fish and Wildlife Service as the Navarre Unit of the Ottawa National Wildlife Refuge. These areas are shown on Figure 11.2-6.

The major station structures are located approximately in the center of the site in areas that have been historically upland. This area of the site has been built up from 6 to 14 ft. above the existing grade to an elevation of 584 ft. International Great Lakes Datum (I.G.L.D.), utilizing glaciolacustrine and till soils taken from three borrow pit areas on other upland portions of the site. These borrow pit locations are shown on Figure 11.2-6. Quarry operations were conducted in one borrow pit area to obtain granular backfill material required during construction. These borrow pits and quarry areas, being exposed to the groundwater aquifer, are filled with water to a level corresponding to the aquifer level, which is approximately mean lake level. An aerial photograph of the site is shown on Figure 2.1-4.

2.1.2.1 Exclusion Area Control

The areas leased to the U. S. Government are, by provisions of the lease, to be operated as a National Wildlife Refuge. Access to these areas is limited to government employees, employees of the Energy Harbor Nuclear Corp. and to others authorized by the government or Energy Harbor Nuclear Corp. Energy Harbor Nuclear Corp. has complete authority to exclude personnel from these areas if station conditions require such exclusion.

Maintenance of the dikes surrounding the marsh areas is the responsibility of Energy Harbor Nuclear Corp. Water level control of the marsh areas is the responsibility of the Fish and Wildlife Service. There is expected to be only limited physical activity in or personnel access to these areas.

2.1.2.2 Boundaries for Establishing Effluent Release Limits

The unrestricted area boundary as shown on Figure 11.2-6 is the boundary line which determines the limits of gaseous releases for purposes of the Offsite Dose Calculation Manual limits to fulfill the requirements of 10CFR20 pertaining to radioactive releases to unrestricted areas. The distance from the vent stack for gaseous releases, located on the side of the Shield Building, to the nearest unrestricted area boundary is 2,400 ft.

The site boundary for accident analysis use is shown on Figure 1.2-12. The site boundary and unrestricted area boundary are the same except for a small area along the southern edge of the site, which borders the Toussaint River. The Offsite Dose Calculation Manual (ODCM) uses the actual property line as the unrestricted area boundary, while the site boundary for accident analysis has used a more conservative boundary.

Liquid releases resulting from station operation, including effluents which can contain radioactivity, are discharged into Lake Erie through a buried discharge line running from the area of the station structures through the marsh and under the lake bottom to a slot type discharge nozzle located approximately 930 ft. from the shoreline at a depth 6 ft. below lake low water datum elevation as shown on Figure 11.2-6.

Station area storm water drainage is collected in a storm drain system and is discharged into a drainage ditch which runs along the southern boundary and then flows into a marsh pond which can, for marsh water level control, be pumped into the Toussaint River. Turbine room and other

non-radioactive area drains may also flow into this storm drain system. This drainage ditch is shown on Figure 11.2.6.

These are the only discharge points for radiological effluents from the station area for liquid discharges. The discharge point in Lake Erie is the main discharge source for radiological liquid effluents.

2.1.3 Population and Population Distribution

Population data presented in this section is based on the 1970 U.S. decennial census of population for Michigan and Ohio and on the 1966 Canadian census for areas within the Province of Ontario, Canada. The Canadian census figures were estimated upward to arrive at a 1970 population based on the growth rate of townships and incorporated areas from the 1961 census to the 1966 census.

2.1.3.1 Population Within Ten Miles

The following information regarding population distribution is for historical purposes only. For current population estimates, refer to Development of Evacuation Time Estimates For The Davis-Besse Nuclear Power Station.

Nearly all of the area within a radius of ten miles from the station falls within Ottawa County, Ohio. The assignment of population to this population rose was accomplished by use of census data for townships and incorporated areas with review of U.S. Geological Survey maps for resident location supplemented by field checks where applicable.

Population projections were taken from the adopted documents of the responsible planning agencies for the three counties in Ohio. See Table 2.1-1. Permanent resident population data for the area within ten miles of the site is shown on Figure 2.1-5 and on Table 2.1-2.

2.1.3.2 Population Between Ten and Fifty Miles

Allocation of population to the various segments of the ten to fifty mile population rose was made by townships and incorporated areas according to the 1970 U.S. decennial census tabulations for Ohio and Michigan.

Corresponding 1970 population data for Canada was estimated using 1961 and 1966 census data as a basis for extrapolation.

Population growth projections for Ohio areas were estimated on a county-wide basis using the migration trend from the 1960 to 1970 census with the projections adjusted for changing birth rates in accordance with Bureau of Census population projection D dated February 1972. See Table 2.1-3.

The source of population for Michigan and Canada was a study of the urban Detroit area conducted by Doxiadis Associates, Wayne State University and The Detroit Edison Company directed by Constantinos Doxiadis. These projections, on a similar county-wide basis, are shown on Table 2.1-3.

Population data based on the 1970 census for the area between 0 and 50 miles from the site is given on Figure 2.1-6. Population projections for this area, by decades, are given on Table 2.1-4.

2.1.3.3 Low Population Zone

The following information regarding population distribution is for historical purposes only. For current population estimates, refer to Development of Evacuation Time Estimates For The Davis-Besse Nuclear Power Station.

The low population zone has been established as that area outside of the site boundary within a radius of two miles from the center of the containment structure. The outer boundary of the low population zone was selected as the distance from the containment at which an individual would receive approximately one-half the radiation dose allowed by 10CFR100 as a consequence of the postulated maximum hypothetical accident which serves as design basis for the station and which is analyzed in Chapter 15.

The population rise for this low population zone is shown on Figure 2.1-7. The population was determined by a physical field check of all dwellings within this area and assignment of the representative household figure consistent with the population census figures for Carroll Township, within which this zone is located.

The increase in population during the summer was estimated from a review of electrical service meter turn-on and turn-off records, survey of certain areas, and general review of housing types in this limited area.

There is a Wild Wing campsite consisting of approximately 400 camper sites and a Green Cove condominium development consisting of approximately 746 units located approximately 1 mile northwest. During peak weekend periods, it is estimated that 1930 persons will be using these facilities.

2.1.3.4 Transient Population

The following information regarding population distribution is for historical purposes only. For current population estimates, refer to Development of Evacuation Time Estimates For The Davis-Besse Nuclear Power Station.

Variations in resident population on a seasonal basis, for the first ten miles, are shown on Tables 2.1-5 and 2.1-6. This increase during the summer period is associated with summer residence and cottage-type housing principally located along the Lake Erie shoreline areas.

Within the low population zone, there is no activity which causes variations in daily distribution other than normal work patterns except that the fullest use of the summer residences is during weekend periods.

The employment at Erie Industrial Park located four miles southeast of the site amounts to approximately 850. The activities at the Camp Perry facility located to the east of Erie Industrial Park result in significant transient variations in population at this facility. This facility is used as a training camp by the Ohio National Guard for weekend training throughout the year and for two-week periods during the summer months. Camp Perry is also the site of the National Rifle Matches. Weekend National Guard training activities result in approximately 200,000 man-days of occupancy during the year, and approximately 2,000 personnel per year are engaged in a two-week period training during the summer months. The National Rifle Matches during the latter part of July and all of August result in an average participation of 3,000 people, most of whom reside at Camp Perry during their attendance.

Total summer population includes permanent resident population as shown on Table 2.1-2 plus summer resort residents who live in cottages or trailers during the summer months only. See Table 2.1-5.

Population estimates for the zero to five-mile area were made by an individual house-to-house count of all dwellings, including summer cottages and trailers, in each segment of the one-mile annular areas. A multiplier in terms of people per household, calculated from data given in the 1970 census for Carroll and Erie townships, was applied to the number of dwellings in each segment to attain corresponding population figures.

The difference between total summer population shown above and permanent resident population as shown on Table 2.1-2 was determined by a review of electrical service meter turn-on and turn-off records, survey of certain areas, and consultation with local residents.

2.1.3.5 Population Centers

The nearest large population center is the City of Toledo located WNW of the site at a distance of 20 miles to the nearest corporation limit. Toledo had a 1970 census population of 383,818. The City of Sandusky, located 20 miles ESE of the site, had a 1970 census population of 32,674. Other population centers located within 50 miles of the station are shown on Table 2.1-7.

2.1.3.6 Public Facilities and Institutions

The following information regarding population distribution is for historical purposes only. For current population estimates, refer to <u>Development of Evacuation Time Estimates For The Davis-Besse Nuclear Power Station</u>.

Public facilities within ten miles of the station site are shown on Figure 2.1-8. The major park facility in this area is the Crane Creek State Park which has an average daily summertime attendance of approximately 2,500 with a peak attendance of approximately 5,000. There are no overnight facilities in the park, and it is closed to visitors at night.

The Magee Marsh State Wildlife area and Sportsmen Migratory Bird Center located there draw an annual attendance of approximately 48,000 with a peak day visitation of about 1,500.

The other park facilities are considerably smaller and have correspondingly smaller attendance. A listing of parks, schools and other public facilities shown of Figure 2.1-8 are given in Table 2.1-8 with size and nature of facility with attendance as appropriate.

2.1.4 <u>Uses of Adjacent Lands and Waters</u>

2.1.4.1 Land Use

The 1970 land use for Ottawa County is given in Table 2.1-9.

2.1.4.1.1 Agriculture

Agriculture is a major source of income in Ottawa County. The major crops of the county include peaches, grapes, apples, corn, wheat, soy beans, oats, hay, tomatoes, pumpkins, and sugar beets. The raising of livestock is not a major activity in this area. Farming activities in the

county are expected to decrease slightly in the future. The agriculture land use for all counties within 50 miles of the station site is shown in Table 2.1-10. The livestock in these counties is shown in Table 2.1-11.

Dairying is not a major activity in this area. The following table gives the location of dairy cows within five miles of the station site:

Distance from Station (miles)	Direction from Station	Number of Cows
2 1/2	WSW	75
3 1/2	SSW	52
4	S	35

The above data on dairy cows was obtained from the Ottawa County Agricultural Agent. Most of the milk from these cows is sold directly to dairies.

The above information is historical; refer to the Offsite Dose Calculation Manual and USAR Section 11.6 for the current requirements of the land use/radiological environmental monitoring program.

2.1.4.1.2 Industry

In 1969 there were 74 manufacturing firms in Ottawa County, of which only nine had over 100 employees. These firms are centered in and around Port Clinton. Major products and activities include gypsum products, rubber products, boat building and repairing, canning, and machinery and metal products. None of these industries has a potentially significant effect on the safe operation of the station, nor will it be adversely affected by the operation of the station.

The closest industrial firms to the station site are located at the Erie Industrial Park, approximately four miles to the southeast. The Erie Industrial park was bought by the Uniroyal Corporation on a lease purchase arrangement with the Ottawa County Community Development Corporation which bought the deactivated depot in 1966. Uniroyal has the largest manufacturing facility at the park (350 employees), and there are nine other smaller firms located in the park. These industries and the service groups for the park employ about 850 people.

2.1.4.1.3 Recreational

This area is one of active duck hunting and sport fishing, and in addition, there is a significant number of resorts on the Lake Erie islands and on Marblehead Peninsula east of Port Clinton. Parks and recreational areas located in Ottawa County are shown in Table 2.1-8.

The marsh areas on the station site, which comprise more than half of the site, are leased to the Fish and Wildlife Service for management as a National Wildlife Refuge.

2.1.4.2 Water Use

Activities on Lake Erie and the streams flowing into it comprise nearly all of the utilizations of the waters in this area. There is a significant amount of both commercial and recreational usages of these waters. Commercial uses include fishing and shipping while recreational uses include sport fishing, boating, and swimming. The total catch of fish from Lake Erie is shown in

Table 2.1-12. Also shown in this table are the catches of fish from commercial fishermen which were taken in waters which can be described as adjacent to Ottawa County.

TABLE 2.1-1

Census Population and Projections by Townships
Within Ten Miles of Station

	Census		Projection			
Township	1960	1970	1980	1990	2000	2010
0 to 5 miles						
Carroll (1)	1,570	1,355	1,350	1,350	1,555	1,690
Erie (1)	1,566	1,470	1,500	1,600	1,650	1,790
5 to 10 Miles						
Bay (1)	1,716	1,798	1,950	2,250	2,400	2,605
Benton (1)	2,366	2,340	2,400	2,750	2,955	3,210
Harris (1)	2,675	2,784	3,000	3,400	3,625	3,940
Jerusalem (2)	3,319	3,405	3,700	4,100	4,450	4,800
Portage (1)	8,111	7,948	8,200	9,300	9,900	10,750
Rice (3)	753	851	980	1,125	1,295	1,450
Salem (1)	4,476	4,508	4,700	5,400	5,785	6,285

Description of basis for population projection

The primary projection sources for the zero to ten-mile area are the adopted documents of the responsible planning agencies for the three major political jurisdictions involved; namely, Lucas, Ottawa and Sandusky counties in Ohio. These sources are as follows:

(1) OTTAWA COUNTY – Volume 1: Population and Economic Study, Ottawa Regional Planning Commission, 1971.

(2) LUCAS COUNTY - Regional Population Distribution, for the Toledo Regional Area, Technical Report 3.13, December 1966, Toledo-Lucas County Planning Commissions.

(3) SANDUSKY COUNTY – Report 1, Sandusky County Comprehensive Plan, July 1961, Carroll V. Hill.

TABLE 2.1-2

Population Within Ten Miles

Permanent Resident Population

Segment	1970	1980	1990	2000	2010
North					
0-1 Mile	11	11	11	13	14
1-10	0	0	0	0	0
Total 0-10	11	11	11	13	14
NNE					
0-1 Mile	30	30	30	35	37
1-10	0	0	0	0	0
Total 0-10	30	30	30	35	37
NE					
0-1 Mile	4	4	4	5	6
1-10	0	0	0	0	0
Total 0-10	4	4	4	5	6
ENE					
Total 0-10	0	0	0	0	0
East					
Total 0-10	0	0	0	0	0
ESE					
Total 0-10	0	0	0	0	0
SE					
0-4 Miles	0	0	0	0	0
4-5	98	100	107	110	119
5-10	1,923	2,003	2,248	2,379	2,583
Total 0-10	2,021	2,103	2,355	2,489	2,702
SSE					
0-1 Mile	0	0	0	0	0
1-2	47	47	47	54	58
2-3	52	52	52	60	65
3-4	70	81	85	89	94
4-5	75	77	81	85	91
5-10	1,324	1,412	1,595	1,687	1,833
Total 0-10	1,568	1,669	1,860	1,975	2,141

TABLE 2.1-2 (Continued)

Population Within Ten Miles

Permanent Resident Population

Segment	1970	1980	1990	2000	2010
South					
0-1 Mile	9	9	9	10	11
1-2	47	47	47	54	58
2-3	71	71	71	82	88
3-4	59	59	59	68	73
4-5	83	84	89	100	107
5-10	2,198	2,318	2,660	2,866	3,128
Total 0-10	2,467	2,588	2,935	3,180	3,465
SSW					
0-1 Mile	21	21	21	24	26
1-2	43	43	43	49	53
2-3	43	43	43	49	53
3-4	64	64	64	74	79
4-5	89	89	92	104	111
5-10	2,316	2,417	2,771	2,973	3,229
Total 0-10	2,576	2,677	3,034	3,273	3,551
SW					
0-1 Mile	16	16	16	19	20
1-2	0	0	0	0	0
2-3	35	35	35	40	44
3-4	59	59	59	71	74
4-5	65	65	65	75	82
5-10	1,302	1,368	1,547	1,665	1,807
Total 0-10	1,477	1,543	1,722	1,870	2,027
WSW					
0-1 Mile	0	0	0	0	0
1-2	10	10	10	12	12
2-3	10	10	10	12	12
3-4	43	43	43	50	50
4-5	60	60	60	69	74
5-10	786	804	917	991	1,074
Total 0-10	909	927	1,040	1,134	1,222

TABLE 2.1-2 (Continued)

Population Within Ten Miles

Permanent Resident Population

Segment	1970	1980	1990	2000	2010
West					
0-1 Mile	19	19	19	22	24
1-2	62	62	62	71	76
2-3	36	36	36	41	45
3-4	57	57	57	66	71
4-5	91	91	91	104	112
5-10	817	837	959	1,036	1,121
Total 0-10	1,082	1,102	1,224	1,340	1,449
WNW					
0-1 Mile	0	0	0	0	0
1-2	33	33	33	38	41
2-3	169	169	169	194	209
3-5	0	0	0	0	0
5-10	803	864	962	1,043	1,125
Total 0-10	1,005	1,066	1,164	1,275	1,375
NW					
0-1 Mile	4	4	4	5	6
1-2	149	149	149	171	184
2-3	43	43	43	49	53
3-10	0	0	0	0	0
Total 0-10	196	196	196	225	243
NNW					
0-1 Mile	67	67	67	78	83
1-2	19	19	19	22	24
2-10	0	0	0	0	0
Total 0-10	86	86	86	100	107
Totals					
0-1 Mile	181	181	181	211	227
1-2	410	410	410	471	506
2-3	459	459	459	527	569
3-4	352	363	367	418	441
4-5	561	566	585	647	696
5-10	11,469	12,023	13,659	14,640	15,900
Total 0-10	13,432	14,002	15,661	16,914	18,339

Note: The locations of Segments included in the above tabulation are shown in Figure 2.1-5.

TABLE 2.1-3

Population Projections by Counties Within 50 Miles of Station Site

	Census Actuals		Projection			
OHIO (1)	1960	1970	1980	1990	2000	2010
Crawford	46,775	50,364	54,040	58,090	64,640	65,400
Erie	68,000	75,909	84,260	93,700	102,320	111,733
Fulton	29,301	33,071	37,140	41,780	46,080	50,826
Hancock	53,686	61,217	69,360	78,790	87,540	97,257
Henry	25,392	27,058	28,760	30,600	32,190	33,864
Huron	47,326	49,587	51,870	54,310	56,370	58,512
Lorain	217,500	256,843	301,790	355,510	407,410	466,892
Lucas	456,931	484,370	512,000	541,670	567,700	594,950
Ottawa	35,323	37,099	38,880	40,750	42,380	44,075
Richland	117,761	129,997	142,870	157,300	170,350	184,409
Sandusky	56,486	60,983	65,620	70,740	75,260	80,077
Seneca	59,326	60,696	62,030	63,400	64,540	65,702
Wood	72,596	89,722	109,820	134,970	160,340	190,484
Wyandot	21,648	21,826	22,000	22,180	22,310	22,444
MICHIGAN (2)(3)						
Lenawee	77,789	81,609	85,500	126,604	187,700	232,916
Monroe	101,120	118,479	340,800	532,617	883,600	1,067,211
Washtenaw	172,440	234,103	416,100	609,648	892,800	1,099,870
Wayne	2,666,297	2,666,751	2,743,000	2,979,995	3,238,300	3,390,675
ONTARIO (4)						
Essex	258,218	305,643	378,400	487,390	627,600	724,738

(1) These projections assume that net migration patterns experienced over the last decade will continue and that birth and death rates will follow national patterns as indicated in the Bureau of the Census projections of the population of the U.S. (See U.S. Department of Commerce, Bureau of the Census (CPR Series Projections of Population of the U.S. by Age and Sex: 1970 to 2020, Nov. 1971).)

(2) The source of these population projections is the “Developing Detroit Area Research Project”; an effort of Doxiadis Associates, Wayne State University, and The Detroit Edison Company, directed by Constantinos Doxiadis.

(3) Projections for the years 1990 and 2010 are interpolations and extrapolations of data provided by Doxiadis Associates for the years 1980 and 2000.

(4) 1960 census figure is actual 1961 census. 1970 census is actual 1966 census figure of 280,922 extrapolated to what would be expected for 1970 based on the increase from 1961 to 1966 census.

TABLE 2.1-4

Population Between Zero and Fifty Miles

Permanent Resident Population

Segment	1970	1980	1990	2000	2010
North					
0-10 Mile	11	11	11	13	14
10-20	0	0	0	0	0
20-30	248	508	764	1,141	1,426
30-40	50,431	58,064	73,248	91,374	103,042
40-50	578,697	631,795	746,552	881,073	967,473
Total 0-50	629,387	690,378	820,575	973,601	1,071,955
NNE					
0-10 Mile	30	30	30	35	37
10-20	0	0	0	0	0
20-30	908	1,252	1,760	2,415	2,865
30-40	17,327	23,894	33,580	46,090	54,667
40-50	18,920	26,091	36,667	50,327	59,693
Total 0-50	37,185	51,267	72,037	98,867	117,262
NE					
0-10 Mile	4	4	4	5	6
10-20	0	0	0	0	0
20-30	54	54	54	54	54
30-40	5,337	7,360	10,343	14,197	16,838
40-50	13,125	18,099	25,436	34,913	41,410
Total 0-50	18,520	25,517	35,837	49,169	58,308
ENE					
0-10 Mile	0	0	0	0	0
10-20	507	532	557	579	602
20-30	479	479	479	479	479
30-40	0	0	0	0	0
40-50	0	0	0	0	0
Total 0-50	986	1,011	1,036	1,058	1,081

TABLE 2.1-4 (Continued)

Population Between Zero and Fifty Miles

Permanent Resident Population

Segment	1970	1980	1990	2000	2010
East					
0-10 Mile	0	0	0	0	0
10-20	1,596	1,678	1,764	1,840	1,919
20-30	97	108	120	131	143
30-40	0	0	0	0	0
40-50	27,099	31,842	37,509	42,985	49,261
Total 0-50	28,792	33,628	39,393	44,956	51,323
ESE					
0-10 Mile	0	0	0	0	0
10-20	10,799	11,987	13,330	14,556	15,895
20-30	41,039	45,554	50,655	55,316	60,405
30-40	15,472	17,174	19,098	20,855	22,774
40-50	100,699	118,100	138,879	158,943	181,923
Total 0-50	168,009	192,815	221,962	249,670	280,997
SE					
0-10 Mile	2,021	2,103	2,355	2,489	2,702
10-20	5,257	5,719	6,233	6,697	7,198
20-30	10,864	11,979	13,236	14,380	15,626
30-40	22,566	23,732	24,992	26,076	27,214
40-50	9,543	9,982	10,451	10,848	11,261
Total 0-50	52,251	53,515	57,267	60,490	64,001
SSE					
0-10 Mile	1,568	1,669	1,860	1,975	2,141
10-20	4,000	4,304	4,640	4,937	5,253
20-30	15,256	16,190	17,210	18,103	19,045
30-40	5,022	5,196	5,378	5,532	5,691
40-50	16,358	17,368	18,470	19,432	20,448
Total 0-50	42,204	44,727	47,558	49,979	52,578

TABLE 2.1-4 (Continued)

Population Between Zero and Fifty Miles

Permanent Resident Population

Segment	1970	1980	1990	2000	2010
South					
0-10 Mile	2,467	2,588	2,935	3,180	3,465
10-20	26,958	29,007	31,270	33,271	35,401
20-30	8,591	9,074	9,602	10,066	10,556
30-40	29,733	30,387	31,056	31,615	32,184
40-50	7,707	8,032	8,384	8,690	9,014
Total 0-50	75,456	79,088	83,247	86,822	90,620
SSW					
0-10 Mile	2,576	2,677	3,034	3,273	3,551
10-20	3,943	4,243	4,574	4,867	5,179
20-30	5,326	5,571	5,835	6,065	6,308
30-40	24,728	27,308	30,277	33,108	36,052
40-50	10,186	10,673	11,226	11,728	12,280
Total 0-50	46,759	50,472	54,946	58,951	63,370
SW					
0-10 Mile	1,477	1,543	1,722	1,870	2,027
10-20	5,860	6,259	6,694	7,079	7,486
20-30	10,134	11,936	14,155	16,361	18,953
30-40	8,928	10,861	13,270	15,691	18,560
40-50	45,329	51,650	59,038	65,964	73,727
Total 0-50	71,728	82,249	94,879	106,965	120,753
WSW					
0-10 Mile	909	927	1,040	1,134	1,222
10-20	8,179	8,935	9,815	10,658	11,614
20-30	17,889	21,896	26,911	31,970	37,981
30-40	32,124	38,960	47,476	56,503	66,214
40-50	9,131	10,418	11,977	13,503	15,279
Total 0-50	68,232	81,136	97,219	113,768	132,310
West					
0-10 Mile	1,082	1,102	1,224	1,340	1,449
10-20	13,322	15,057	17,124	19,159	21,501
20-30	231,673	246,625	263,278	278,236	294,476
30-40	38,667	41,915	45,436	48,753	52,264
40-50	16,079	17,975	20,326	22,621	25,036
Total 0-50	300,823	322,674	347,388	370,109	394,726

TABLE 2.1-4 (Continued)

Population Between Zero and Fifty Miles

Permanent Resident Population

Segment	1970	1980	1990	2000	2010
WNW					
0-10 Mile	1,005	1,066	1,164	1,275	1,375
10-20	11,664	12,644	13,706	14,707	15,766
20-30	203,406	243,851	280,802	331,186	372,660
30-40	36,725	68,171	95,775	137,607	170,682
40-50	10,656	11,164	16,500	24,461	30,301
Total 0-50	263,456	336,896	407,947	509,236	590,784
NW					
0-10 Mile	196	196	196	225	243
10-20	0	0	0	0	0
20-30	37,635	107,900	168,647	263,595	337,670
30-40	12,968	37,179	58,111	90,828	116,351
40-50	15,287	32,920	50,019	76,078	95,918
Total 0-50	66,086	178,195	276,973	430,726	550,182
NNW					
0-10 Mile	86	86	86	100	107
10-20	0	0	0	0	0
20-30	12,854	36,968	57,781	90,312	115,780
30-40	32,769	60,684	87,518	127,685	158,242
40-50	153,120	164,354	196,850	232,834	254,428
Total 0-50	198,829	262,092	342,235	450,931	528,557
Totals					
0-10 Mile	13,432	14,002	15,661	16,914	18,339
10-20	92,085	100,365	109,707	118,350	127,814
20-30	596,453	759,945	911,289	1,119,810	1,294,427
30-40	322,797	450,885	575,558	745,824	880,775
40-50	1,031,936	1,160,463	1,388,284	1,654,000	1,847,452
Total 0-50	2,066,703	2,485,660	3,000,499	3,655,298	4,168,807

Note: The locations of Segments included in the above tabulation are shown on Figure 2.1-6.

TABLE 2.1-5

Seasonal Resident Population Variations Within Ten Miles

Estimated Increases in Summer Population for
Areas Affected by Variations on a Seasonal Basis

Segment	1970	1980	1990	2000	2010
North					
0-1 Mile	33	33	33	38	41
Total 0-10	33	33	33	38	41
NNE					
0-1 Mile	40	40	40	48	51
Total 0-10	40	40	40	48	51
NE					
0-1 Mile	10	10	10	11	12
Total 0-10	10	10	10	11	12
SE					
1-2 Miles	100	100	100	116	124
5-10	1,730	1,828	2,064	2,187	2,375
Total 0-10	1,830	1,928	2,164	2,303	2,499
South					
0-1 Mile	159	159	159	184	197
3-4	64	65	65	75	81
Total 0-10	224	224	224	259	278
SSW					
0-1 Mile	243	243	243	280	301
4-5	80	80	83	91	99
Total 0-10	323	323	326	371	400
SW					
0-1 Mile	57	57	57	65	71
Total 0-10	57	57	57	65	71
West					
4-5 Miles	94	94	94	109	117
Total 0-10	94	94	94	109	117
WNW					
1-2 Miles	100	100	100	115	123
2-3	506	506	506	583	627
Total 0-10	606	606	606	698	750

TABLE 2.1-5 (Continued)

Seasonal Resident Population Variations Within Ten Miles

Segment	1970	1980	1990	2000	2010
NW					
1-2 Miles	379	379	379	437	470
2-3	79	79	79	91	98
Total 0-10	458	458	458	528	568
WNW					
0-1 Mile	183	183	183	210	226
1-2	63	63	63	72	77
Total 0-10	246	246	246	282	303
Totals					
0-1 Mile	725	725	725	836	899
1-2	642	642	642	740	794
2-3	585	585	585	674	725
3-4	65	65	65	75	81
4-5	174	174	177	200	216
5-10	1,730	1,828	2,064	2,187	2,375
Total 0-10	3,921	4,019	4,258	4,712	5,090

Note: The above figures must be added to the corresponding population figures given in Table 2.1-2 to obtain total summer population for any given segment.

TABLE 2.1-6

Total Estimated Summer Resident Population
Distribution Including Seasonal Increases

0-10 Miles From Station

Annuli Totals	1970	1980	1990	2000	2010
0-1 Mile	906	906	906	1,047	1,126
1-2	1,052	1,052	1,052	1,211	1,300
2-3	1,044	1,044	1,044	1,201	1,294
3-4	417	428	432	493	522
4-5	735	740	762	847	912
5-10	13,199	13,851	15,723	16,827	18,275
Cumulative					
0-1 Mile	906	906	906	1,047	1,126
0-2	1,958	1,958	1,958	2,258	2,426
0-3	3,002	3,002	3,002	3,459	3,720
0-4	3,419	3,430	3,434	3,952	4,242
0-5	4,154	4,170	4,196	4,799	5,154
0-10	17,353	18,021	19,919	21,626	23,429

TABLE 2.1-7

Population Centers Within 50 Miles

	Distance &		Census Actuals		Estimated			
OHIO	Direction		1960	1970	1980	1990	2000	2010
Bowling Green	33 MI.	WSW	13,574	21,760	28,533	37,792	48,102	60,955
Findlay	45 MI.	SW	30,344	35,800	40,229	45,698	51,649	57,382
Fremont	17 MI.	S	17,573	18,490	19,686	21,222	22,578	24,023
Lorain	46 MI.	E	68,932	78,185	87,519	103,098	118,149	130,730
Maumee	29 MI.	W	12,063	15,937	19,443	22,943	26,384	29,022
Oregon	18 MI.	WNW	13,319	16,563	19,213	21,519	23,671	25,565
Sandusky	20 MI.	ESE	31,989	32,674	35,389	39,354	41,951	45,811
Sylvania*	30-40 MI.	WNW	5,187	12,031	12,678	13,500	14,246	15,127
Tiffin	33 MI.	S	21,478	21,596	22,331	22,824	23,234	23,653
Toledo	20 MI.	WNW	379,133	383,818	400,010	419,560	435,930	452,010
MICHIGAN								
Allen Park*	40-50 MI.	N	37,494	40,747	40,784	40,861	40,931	41,013
Dearborn*	40-50 MI.	N	112,007	104,199	104,354	104,551	104,729	104,940
Dearborn Hgts.*	40-50 MI.	N	--------	80,069	80,177	80,329	80,466	80,628
Detroit*	40-50 MI.	N	1,670,144	1,511,336	1,513,691	1,516,545	1,519,139	1,522,200
Inkster*	40-50 MI.	NNW	39,097	38,595	38,645	38,717	38,784	38,862
Lincoln Park*	40-50 MI.	N	53.933	52,984	53,059	53,159	53,250	53,358
Monroe	26 MI.	N	22,968	23,894	39,185	41,928	44,983	48,000

TABLE 2.1-7 (Continued)

Population Centers Within 50 Miles

MICHIGAN	Distance & Direction		Census Actuals 1960	1970	Estimated 1980	1990	2000	2010
Southgate*	40-50 MI.	N	29,404	33,909	33,938	34,002	34,060	34,128
Trenton*	30-40 MI.	N	18,439	24,127	24,150	24,195	24,236	24,285
Westland*	40-50 MI.	NNW	--------	86,749	86,863	87,027	87,176	87,352
Wyandotte	41 MI.	N	43,519	41,061	54,685	56,872	59,780	62,171
ONTARIO								
Windsor	50 MI.	N	114,363	114,741	115,120	117,416	119,863	122,406

OHIO CITIES

Projections are based on the cities maintaining their share of the county population that existed in the 1960-70 decade, with adjustments made for changes in net migrations.

MICHIGAN AND ONTARIO CITIES

The source of these population projections was the "Developing Detroit Area Research Project", an effort of Doxiadis Associates, Wayne State University, and The Detroit Edison Company, directed by Constantinos Doxiadis.

* These population projections have been prepared by Barry Beckham of the University of Toledo.

** These projections assume that each city's share of its county's population in 1970 remained constant for each of the projected county populations to 2010.

TABLE 2.1-8

Public Facilities and Institutions

I.D. No.	Public Facilities Name	Operating Agency	Township	Distance From Station (Miles)	Direction from Station	Area (Acres)	Existing Facilities
(1)	Ottawa National Wildlife Area	U.S. Fish and Wildlife Service	Benton-Jerusalem	7 (4-9)	WNW	4593	Wildlife Refuge
(2)	Ottawa National Wildlife Area Darby Marsh	U.S. Fish and Wildlife Service	Erie	6½ (6-7)	SE	480	Wildlife Refuge
(3)	West Sister National Wildlife Area	U.S. Fish and Wildlife Service	Jerusalem	10	NNW	75	Wildlife Refuge
(4)	Crane Creek State Park	Ohio Department of Natural Resources	Jerusalem-Benton	6 (3-6)	WNW	72	Beach & Swimming, Picnicking
(5)	Little Portage River Wildlife Area	Ohio Department of Natural Resources	Bay	7	SSE	357	Hunting, Fishing
(6)	Toussaint Creek Wildlife Area	Ohio Department of Natural Resources	Carroll	3½	WSW	236	Boat Ramp, Fishing, Hunting
(7)	Magee Marsh Wildlife Area	Ohio Department of Natural Resources	Carroll-Jerusalem	4 (2½-5½)	WNW	2131	Hunting
(8)	Metzger Marsh Wildlife Area	Ohio Department of Natural Resources	Jerusalem	8½ (8-9)	WNW	558	Boat Ramp, Fishing
(9)	Portage River Boat Park Access	Ohio Department of Natural Resources	Bay	8	SE	10	Boat Ramp, Fishing

TABLE 2.1-8 (Continued)

Public Facilities and Institutions

I.D. No.	Public Facilities Name	Operating Agency	Township	Distance From Station (Miles)	Direction from Station	Area (Acres)	Existing Facilities
(10)	Ottawa County Fairgrounds	Ottawa County	Salem	6	S	55	Rack Track, Annual County Fair
(11)	Veterans Memorial Park	Village of Oak Harbor	Salem	6½	SSW	10	Picnicking, Baseball Playground
(12)	Oak Harbor Park	Village of Oak Harbor	Salem	6½	SSW	20	Baseball, Tennis, Field Games
(13)	Waterworks Park	City of Port Clinton	Portage	9½	SE	20	Fishing, Baseball, Picnicking
(14)	Graytown Park	Benton Township	Benton	10	WSW	5	Picnicking, Play Equipment, Field Games
(15)	Oak Harbor High School	Benton-Carroll-Salem Local School District	Salem	6½	SSW	809	
(16)	R.C. Waters Elementary School	Benton-Carroll-Salem Local School District	Salem	6½	SSW	725	
(17)	Carroll Elementary School	Benton-Carroll-Salem Local School District	Carroll	3½	SW	240	
(18)	Rocky Ridge Elementary School	Benton-Carroll-Salem Local School District	Benton	8	SW	215	

Note: Source for (1) through (14) – “A Statewide Plan for Outdoor Recreation in Ohio, 1971-1977,” State of Ohio, Department of Natural Resources, Section II, Lake Plains Region.

TABLE 2.1-8 (Continued)

Public Facilities and Institutions

I.D. No.	Public Facilities Name	Operating Agency	Township	Distance From Station (Miles)	Direction from Station	Area (Acres)	Existing Facilities
(19)	Graytown Elementary School	Benton-Carroll-Salem Local School District	Benton	10	WSW	279	
(20)	Port Clinton High School	Port Clinton Public Schools	Portage	10	SE	1050	
(21)	Bataan Elementary School	Port Clinton Public Schools	Portage	9½	SE	560	
(22)	Erie Elementary School	Port Clinton Public Schools	Erie	6	SSE	135	
(23)	Jefferson Elementary School	Port Clinton Public Schools	Portage	10	SE	550	
(24)	Port Clinton Junior High School	Port Clinton Public Schools	Portage	10	SE	600	
(25)	Kindergarten Building	Port Clinton Public Schools	Portage	9½	SE	146	
(26)	Riverview School for Children	Ottawa County Board of Retardation	Salem	6	S	43	
(27)	St. Boniface Catholic School	St. Boniface Catholic Church-Oak Harbor	Salem	6½	SSE	106	

TABLE 2.1-9

Ottawa County Land Use

Land Use Categories	Acres 1970	Acres Projected Increase (3) Thru 1995
Agriculture	123,942	
Forests and Woodlands (1)	1,079	
Water and Wetlands	1,067	
Residential	9,634	1,812
Institutional	990	72
Transportation	4,679	1,159
Extractive	2,432	
Commercial	1,455	556
Recreation	13,526	128
Utilities (2)	2,363	59
Industrial	2,007	104

NOTES: (1) Excludes areas within designated federal and state recreational and wildlife areas.

(2) Includes utilities and railroad right-of-way and airport areas.

(3) This projected increase in certain categories would come principally from a reduction of agriculture use category. The only projected category change for Carroll Township, the township where the station site is located, is an increase of 25 acres for residential and local road use through 1995.

Source: Ottawa County Comprehensive Planning Program, Volume 2, Regional Development Plan.

TABLE 2.1-10

Agricultural Land Use in Counties Within Fifty Miles of Station Site

Land Use - Acres

	Corn	Wheat	Soybeans	Hay	Alfalfa	Vege-tables	Other Small Grains Including Oats
OHIO							
Crawford	51,604	20,883	52,833	12,491	3,457	13	10,991
Erie	19,851	10,810	17,174	5,130	3,271	3,946	4,636
Fulton	79,678	17,326	50,984	9,038	6,043	2,834	6,529
Hancock	71,726	31,970	94,757	10,910	4,736	1,292	14,832
Henry	67,137	26,306	78,233	9,099	6,726	3,888	10,060
Huron	43,501	22,253	63,816	12,236	4,237	3,502	12,551
Lorain	20,763	9,895	25,243	17,973	6,420	1,555	7,100
Lucas	22,925	7,628	31,038	3,415	2,577	3,653	2,760
Ottawa	11,409	13,109	37,348	12,058	8,840	2,827	5,939
Richland	25,342	11,798	20,633	18,346	5,006	311	10,114
Sandusky	47,312	20,595	54,651	12,423	8,406	7,254	8,237
Seneca	60,449	31,221	81,916	16,685	7,231	1,694	13,710
Wood	89,324	40,787	103,803	18,795	14,811	3,336	20,604
Wyandot	48,618	22,675	61,243	9,763	3,016	374	9,234
Total Ohio	659,639	287,256	773,672	168,362	84,777	36,479	137,297
MICHIGAN							
Lenawee	89,719	31,343	78,292	21,768	13,683	1,340	15,532
Monroe	42,786	22,684	70,220	6,307	3,733	4,899	9,283
Washtenaw	44,590	15,489	11,439	36,136	22,500	1,929	14,486
Wayne	4,723	2,177	11,537	2,643	1,193	2,174	1,258
Total Michigan	181,818	71,693	171,488	66,854	41,109	10,342	40,559
ONTARIO							
Essex*	103,900	45,900	86,500	0	20,300	29,861	19,000
TOTAL FIFTY MILES	945,357	404,849	1,031,660	235,216	146,186	76,682	196,856

Source: U.S. Department of Commerce; 1969 Census of Agriculture.
*Ontario Department of Agriculture and Food, Essex Ontario Extension Branch.

TABLE 2.1-10 (Continued)

Agricultural Land Use in Counties Within Fifty Miles of Station Site

Land Use - Acres

	Sugar Beets	Fruits	Pasture Idle & Misc	Value of Crops Sold (Dollars)	Total in Agriculture	Percent of County in Agriculture
OHIO						
Crawford	0	46	55,000	6,714,844	238,000	91.9
Erie	795	1,305	34,500	4,863,262	107,000	63.2
Fulton	134	124	57,000	10,301,631	204,000	92.1
Hancock	2,542	129	75,000	13,649,800	295,000	98.8
Henry	4,974	22	54,000	15,087,873	266,000	99+
Huron	0	374	82,000	10,745,308	278,000	87.5
Lorain	0	1,360	58,000	10,107,591	162,000	51.1
Lucas	2,173	612	19,000	9,641,418	99,000	44.8
Ottawa	3,803	1,741	37,000	6,211,834	130,000	77.9
Richland	0	831	81,000	3,380,721	200,000	63.0
Sandusky	7,618	1,409	65,500	13,187,931	241,000	92.0
Seneca	1,928	24	90,000	11,562,345	330,000	93.5
Wood	5,369	69	74,000	18,201,980	371,000	93.7
Wyandot	0	138	68,000	8,643,266	242,000	93.1
Total Ohio	29,336	8,184	850,000	142,299,804	3,199,000	
MICHIGAN						
Lenawee	2,098	719	131,500	13,426,625	404,000	63.8
Monroe	3,214	503	76,000	13,695,454	254,000	71.2
Washtenaw	0	773	106,000	5,292,786	260,000	57.2
Wayne	0	469	19,000	4,865,940	50,000	12.8
Total Michigan	5,312	2,464	344,500	37,280,805	968,000	
ONTARIO						
Essex*	1,026	2,840	58,173		367,500	81.2
TOTAL FIFTY MILES	35,674	13,488	1,242,673		4,534,500	

Source: U.S. Department of Commerce; 1969 Census of Agriculture.
*Ontario Department of Agriculture and Food, Essex Ontario Extension Branch.

TABLE 2.1-11

Livestock in Counties Within 50 Miles of Station Site

	Cattle & Calves	Dairy Cows	Hogs	Sheep & Lambs	Chickens	Value of Livestock & Poultry Sold ($)
OHIO						
Crawford	21,431	3,141	51,297	24,986	122,295	10,863,175
Erie	8,212	2,915	7,108	2,489	71,492	4,143,164
Fulton	39,548	4,029	71,393	2,922	765,571	25,326,704
Hancock	18,124	2,848	42,418	2,028	212,231	10,097,714
Henry	13,744	2,990	23,026	4,103	567,667	10,775,731
Huron	15,216	4,228	25,242	16,114	59,510	7,023,631
Lorain	16,865	7,566	7,758	5,063	82,554	6,953,590
Lucas	3,968	267	10,470	421	113,068	2,739,178
Ottawa	5,645	1,524	5,643	1,040	141,914	3,035,428
Richland	21,209	4,523	20,136	12,317	100,543	8,579,858
Sandusky	18,801	3,973	21,959	6,465	138,034	8,020,122
Seneca	19,352	5,818	38,744	22,911	107,028	9,276,912
Wood	23,376	1,622	23,093	7,160	110,556	10,053,135
Wyandot	13,145	2,321	33,848	22,015	113,338	6,886,209
Total Ohio	238,636	47,765	382,135	130,034	2,705,801	123,774,551
MICHIGAN						
Lenawee	46,691	8,771	38,036	12,765	289,562	18,053,016
Monroe	13,984	2,190	15,408	4,441	115,020	6,316,803
Washtenaw	33,588	10,550	23,890	53,361	127,175	13,096,909
Wayne	2,328	537	1,584	500	32,862	992,918
Total Michigan	96,591	22,048	78,918	71,067	564,619	38,459,646
ONTARIO						
Essex*	23,400	17,500	26,500	900	544,000	
TOTAL 50 MILES	358,627	87,313	487,553	202,001	3,814,420	

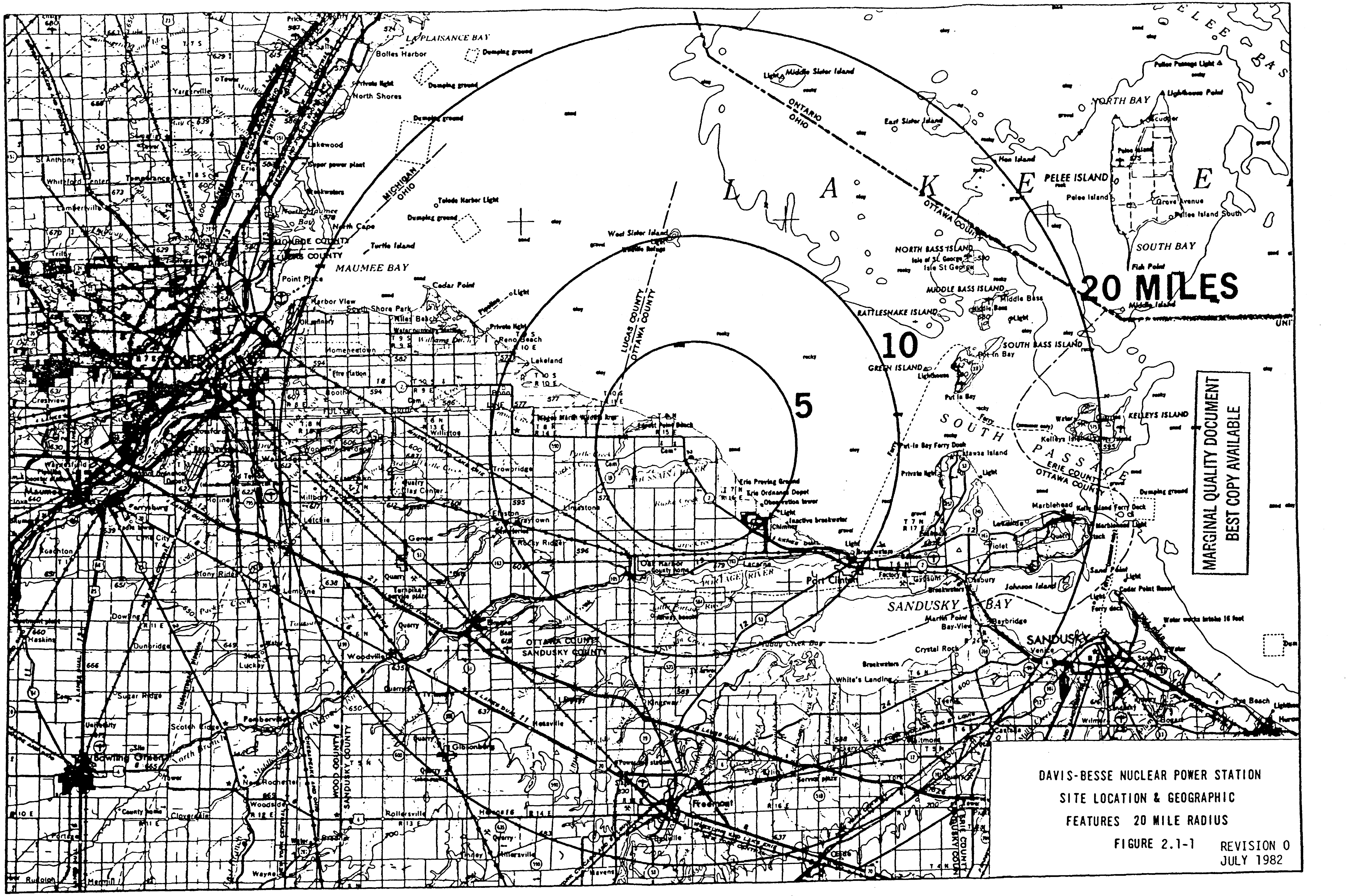
Source: U.S. Department of Commerce; 1969 Census of Agriculture
*Ontario Department of Agriculture and Food, Essex Ontario Extension Branch

TABLE 2.1-12

Fish Landings - 1970

SPECIES	COMMERCIAL FISHING (1) Lake Erie Catch (Lb)	Port Clinton Landings (Lb)	Sandusky Bay Landings (Lb)	SPORT FISHING (2) Lake Erie Total Catch (Lb)
Buffalo	12,163	538	7,267	-
Bullheads	25,198	1,464	17,164	-
Burdot	2	2	0	-
Carp	3,042,210	325,223	1,470,322	-
Catfish	526,718	180,357	176,898	75,000
Gizzard Shad	4,909	570	2,350	-
Goldfish	162,892	6,500	156,390	-
Quillback	21,128	0	0	-
Freshwater Drum	1,037,682	17,552	659,822	125,000
Smelt	259	0	0	-
Suckers	103,832	6,746	43,326	-
White Bass	1,091,864	40,435	294,970	360,000
Perch	2,380,873	10,173	32,238	12,000,000
Walleye	10,258	159	7,998	210,000
Crappies	-	-	-	150,000
Smallmouth Bass	-	-	-	55,000
TOTAL	8,419,988	589,719	2,868,745	12,975,000

Source: (1) Commercial Fish Landings Lake Erie-1970, Ohio Department of Natural Resources, Division of Wildlife, Publication 200.
(2) Estimate of Ohio Division of Wildlife, Letter, R.L. Scholl to L. Roe, October 22, 1971.



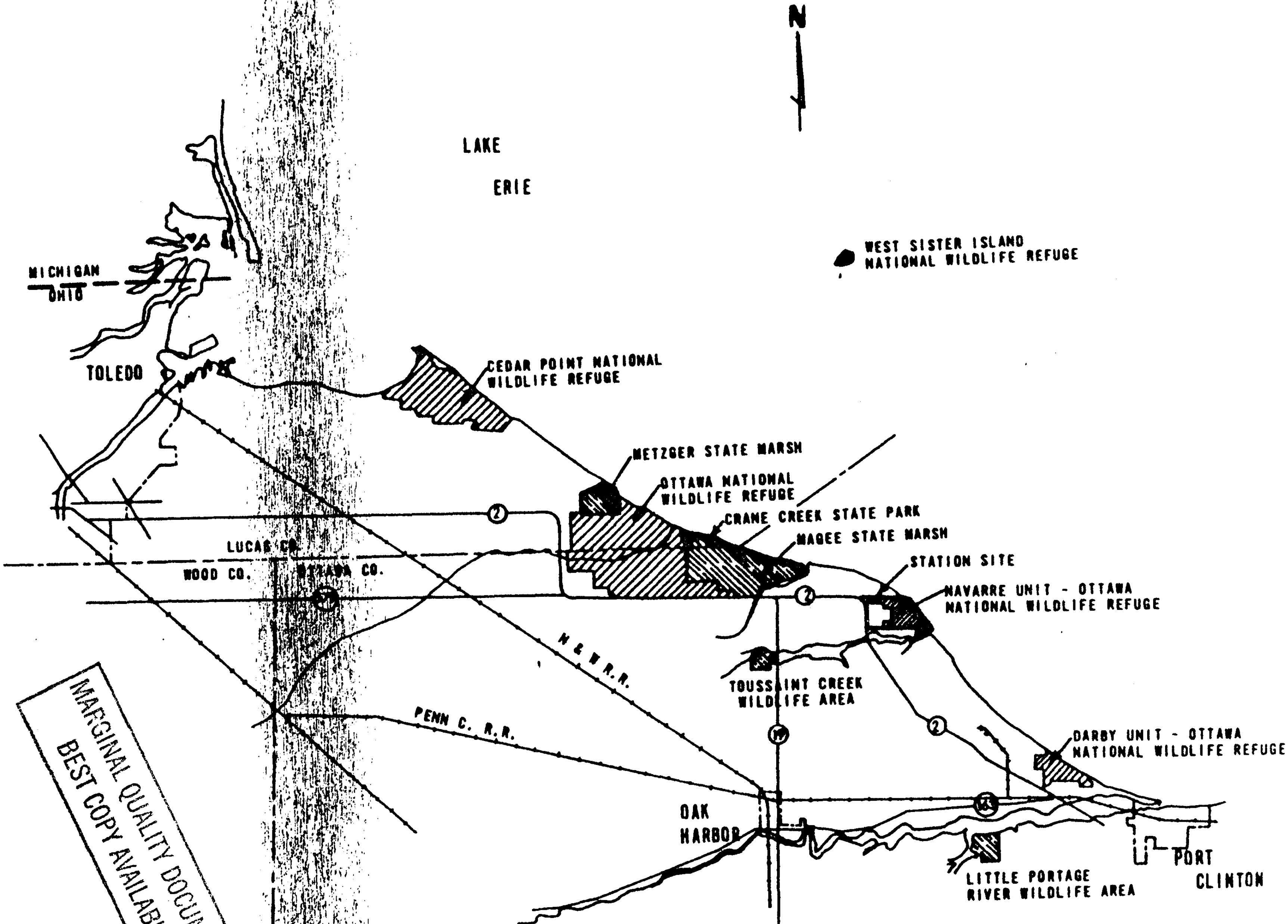
DAVIS-BESSE NUCLEAR POWER STATION

SITE LOCATION & GEOGRAPHIC

FEATURES 20 MILE RADIUS

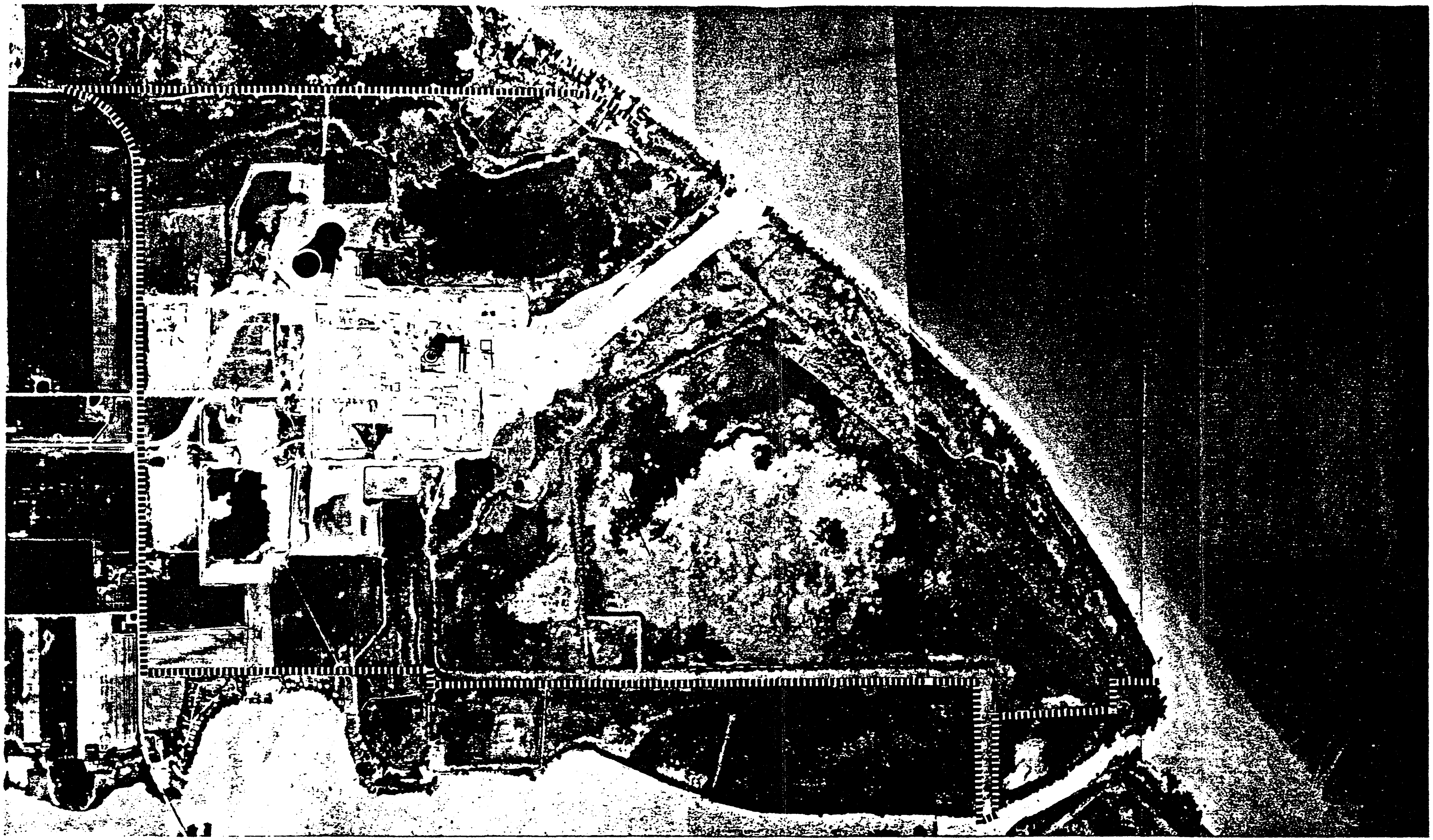
FIGURE 2.1-1 REVISION 0

JULY 1982



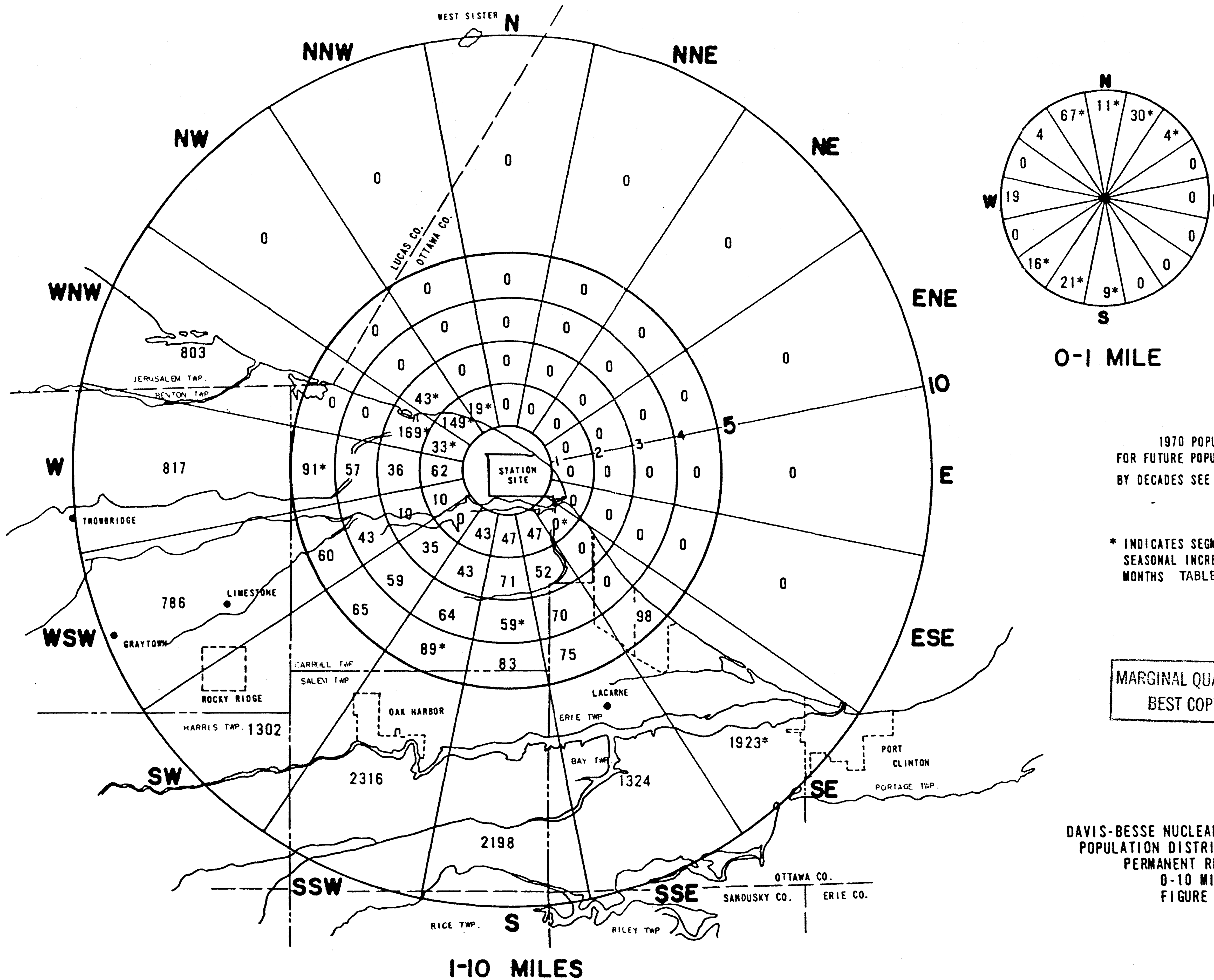
DAVIS-BESSE NUCLEAR POWER STATION
FEDERAL AND STATE WILDLIFE AREAS
FIGURE 2.1-2

REVISION 0
JULY 1982



DAVIS-BESSE NUCLEAR POWER STATION
AERIAL PHOTOGRAPH
SEPT. 4, 1972
FIGURE 2.1-4

REVISION 0
JULY 1982



1970 POPULATION SHOWN
FOR FUTURE POPULATION PROJECTIONS
BY DECADES SEE TABLE 2.1-2

* INDICATES SEGMENT SUBJECT TO SEASONAL INCREASE DURING SUMMER MONTHS TABLE 2.1-5

DAVIS-BESSE NUCLEAR POWER STATION
POPULATION DISTRIBUTION - 1970
PERMANENT RESIDENTS
0-10 MILES
FIGURE 2.1-5

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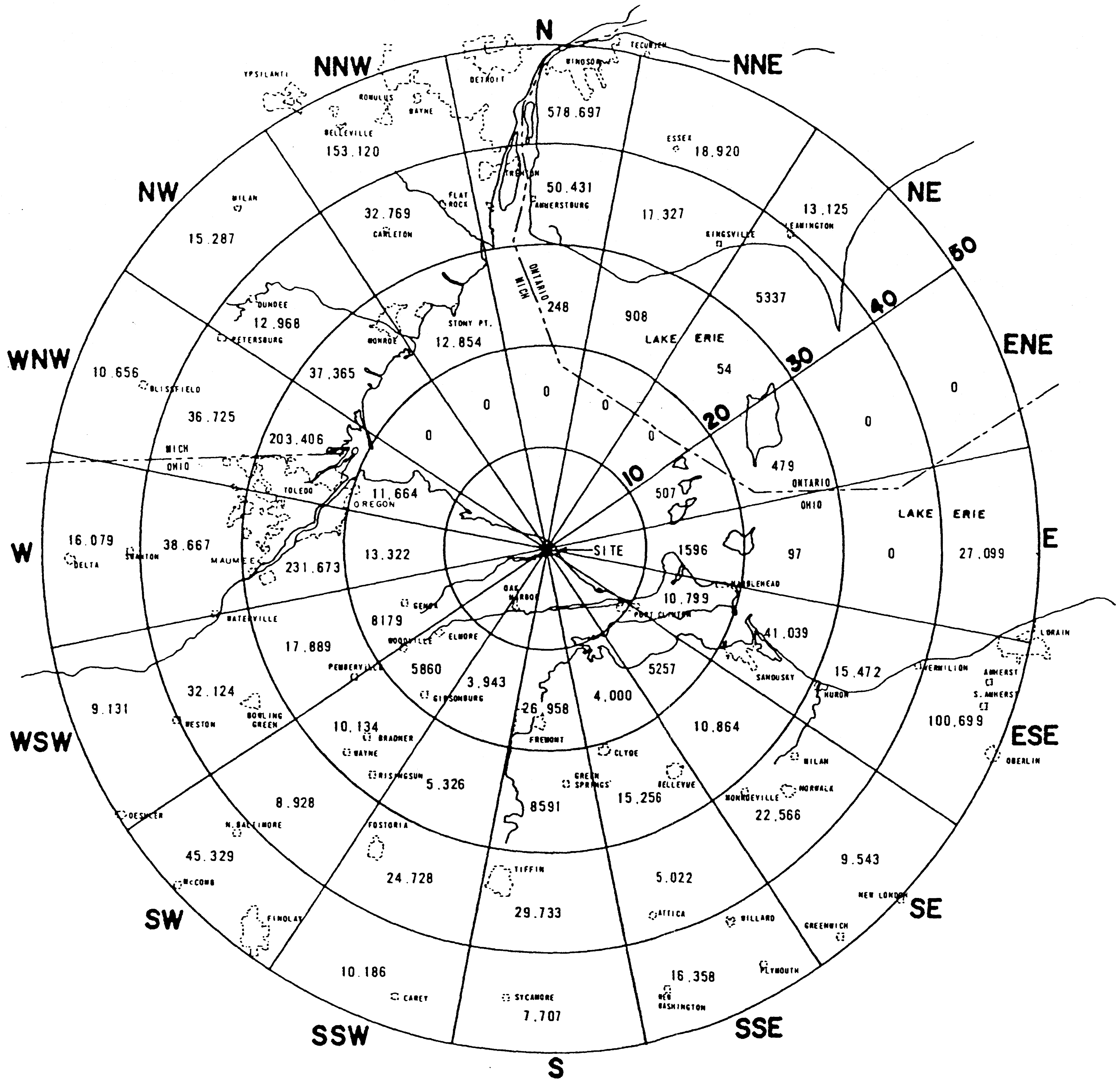
JULY 1982



10-50 MILES

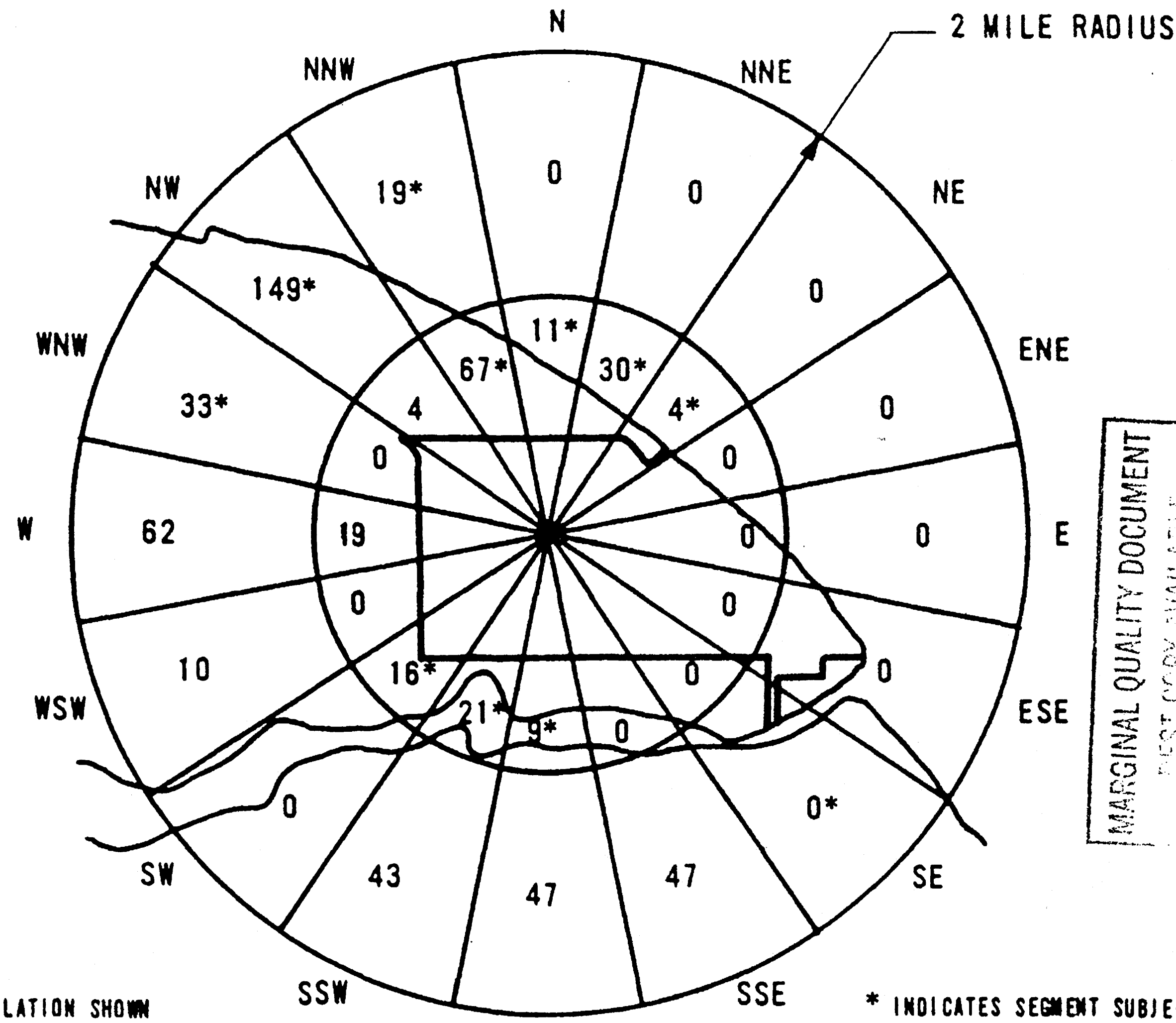
FOR FUTURE POPULATION PROJECTIONS BY DECADES SEE TABLE 2.1-4

CURRENT RESIDENT POPULATION DISTRIBUTION BASED ON 1970 CENSUS.

FOR CURRENT RESIDENT POPULATION DISTRIBUTION 0-10 MILES SEE FIGURE 2.1-5

DAVIS-BESSE NUCLEAR POWER STATION
POPULATION DISTRIBUTION - 1970
10-50 MILES
FIGURE 2.1-6

REVISION 0
JULY 1982



1970 POPULATION SHOWN

* INDICATES SEGMENT SUBJECT TO SEASONAL INCREASE DURING SUMMER MONTHS
SEE Table 2.1-5

PERMANENT POPULATION PROJECTION

ANNULI TOTALS	1970	1980	1990	2000	2010
0-1	181	181	181	211	277
1-2	410	410	410	471	506
TOTAL	591	591	591	682	733

FOR SEGMENT DISTRIBUTION IN FUTURE YEARS SEE Table 2.1-2

FOR SEASONAL INCREASE IN RESIDENT POPULATION DURING SUMMER MONTHS, SEE Table 2.1-5

DAVIS-BESSE NUCLEAR POWER STATIO
LOW POPULATION ZONE
FIGURE 2.1-7

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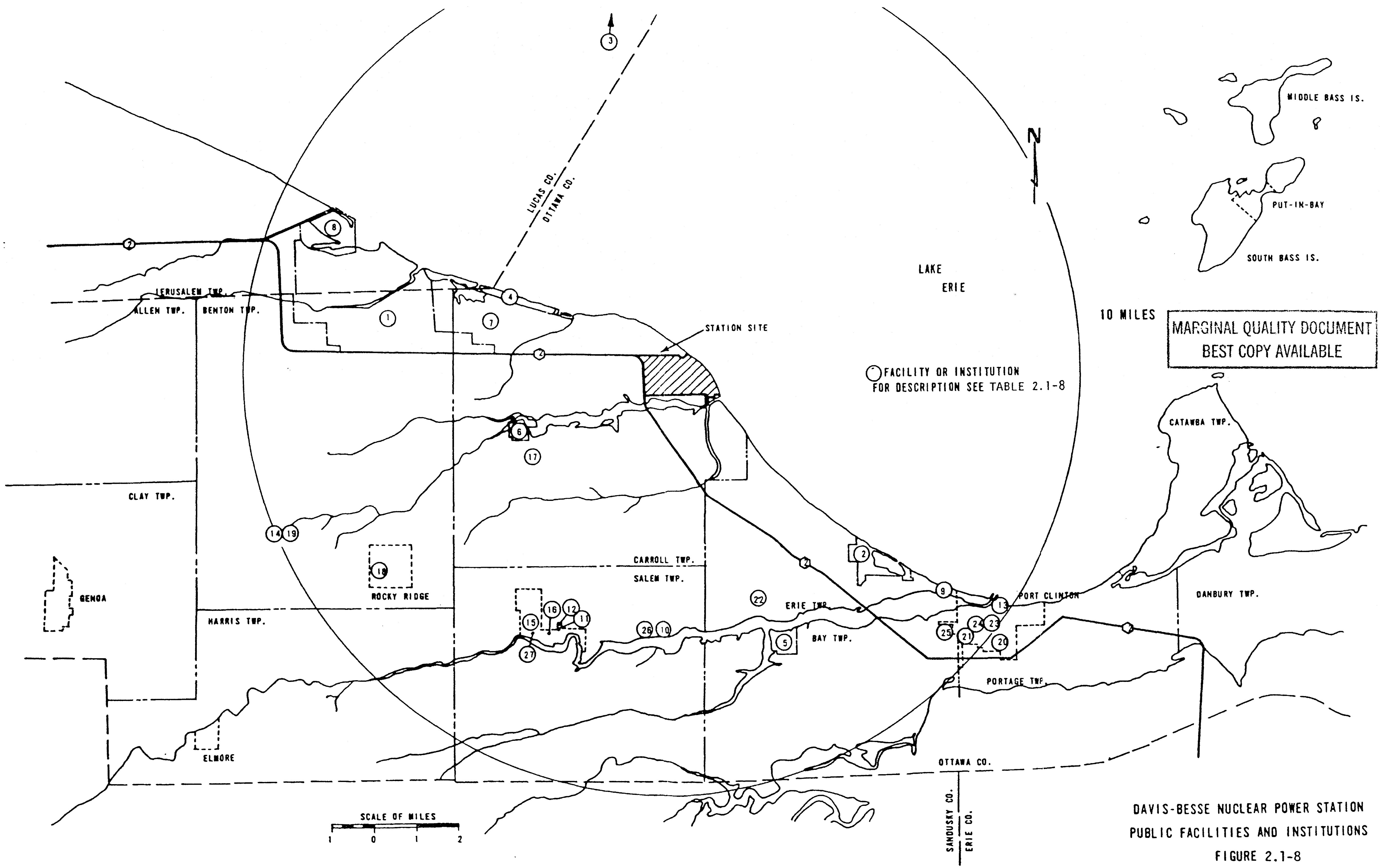
JULY 1982



DAVIS-BESSE NUCLEAR POWER STATION
PUBLIC FACILITIES AND INSTITUTIONS
FIGURE 2.1-8

REVISION 0
JULY 1982

2.2 NEARBY INDUSTRIAL, TRANSPORTATION, AND MILITARY FACILITIES

2.2.1 Locations and Routes

Figure 2.2-1 shows the location of transportation routes, manufacturing plants, military installations, and restricted lake areas in the vicinity of the station site. There are no significant oil or gas pipelines within five miles of the site.

2.2.2 Descriptions

2.2.2.1 Transportation

State Highway Route 2 is located immediately adjacent to the westerly site boundary. This is a two-lane, medium to heavily traveled highway which is used extensively by commercial truck carriers. Approximately six miles east of the site and continuing to the east, this highway becomes a four-lane, divided and limited access highway. Future planning (ref. 33) calls for a westward extension of the limited access construction on an alignment running more than five miles south of the site.

Lake Erie is used extensively for commercial shipping, both domestic and overseas. The shallowness of the western lake basin, particularly near shore, prevents any closer approach than eight miles for ships of any size. The major shipping lanes are approximately 20 miles from the site.

The nearest railroad is the Penn Central which runs in an east-west direction five miles south of the site. The Norfolk & Western Railroad runs in a NW-SE direction from Oak Harbor about six miles southwest of the site. The rail spur serving the station is from the Norfolk & Western main line at a point 7-1/2 miles southwest of the site. This entire spur line is owned by Toledo Edison and has been built solely for service to the Davis-Besse Station.

2.2.2.2 Industrial Facilities

The only industrial facilities located in the vicinity of the site are at the Erie Industrial Park located approximately 4-1/2 miles southeast of the site (Figure 2.2-1). The industrial and commercial activities located in this area are also shown on this figure. Hazardous materials are processed or stored at this location in limited amounts. Ordnance ammunition in an amount equivalent of 10, 000 lbs. of high explosive are stored at Ares, Inc. in approved bunkers.

A public supply water tower has been constructed southwest of the site, adjacent to the site boundary. The tower has been evaluated to have no impact on the meteorological tower operability or plant operation.

2.2.2.3 Aircraft Activities

The closest airport serving commercial airlines is Toledo Express Airport located 38 miles west of the station site. The nearest airport with a paved runway is at Port Clinton located east-southeast, 13 miles from the site.

The two nearest VHF Omni-Directional Radio Range Airways are designated V232 which runs WNW and ESE approximately seven miles south of the station site and V-45 which runs east and west the same distance south of the site.

2.2.2.4 Military Facilities

The Camp Perry Military Reservation is an Ohio National Guard training center located 4.5 miles southeast of the station site immediately adjacent to the east of the Erie Industrial Park as shown on Figure 2.2-1. This installation is used extensively by the Ohio National Guard for training including small arms firing. The limited firing of 40 mm anti-aircraft ordnance was suspended in 1988.

This facility is also the site of the annual National Rifle Matches during July and August of each year.

2.2.2.5 Restricted Lake and Airspace Areas

A portion of Lake Erie including the shoreline of the Erie Industrial Park and Camp Perry has historically been established as a restricted area by the U.S. Army, Corps of Engineers, to provide an impact area for firing of ordnance from Camp Perry and the Erie Ordnance Depot which was deactivated in 1967 and is now the area occupied by the Erie Industrial Park. A reduced lake restricted area exists as shown on Figure 2.2-1. This restricted area was used as an impact area for the anti-aircraft training firing from Camp Perry described in Subsection 2.2.2.4. Anti-aircraft training firing was suspended in 1988.

This lake area is used as an impact area for ordnance test firing by ARES Inc. located in the Erie Industrial Park through a joint use agreement with the Ohio National Guard.

The Federal Aviation Agency has established restricted air space R-5502 over this surface area to prohibit the use of the airspace to low-flying aircraft during ordnance and small arms firing.

A detailed study of the use of the restricted areas is given in Appendix 2A.

2.2.3 Evaluations

2.2.3.1 Transportation

Transportation accidents on State Highway Route 2 could involve trucks carrying flammables, explosives or toxic materials. The arrangement of the entrance roadways, rail line, and site topography adjacent to this highway is such that it precludes a vehicle from accidentally traveling for any distance onto the site from the highway. Evaluations of transported materials and transportation frequencies have demonstrated that fire, explosion or toxic material releases resulting from a truck accident will not pose a safety hazard for control room habitability. The distance of the station structures from the highway is 2,600 feet.

Drainage of this portion of the site is away from the site area containing the station structures which is 6 to 14 feet above natural site elevations. This precludes the spread of burning flammables to the station buildings.

No credible highway transportation accident could adversely affect the safety of the station.

2.2.3.2 Industrial Facilities

None of the manufacturing or warehousing operations at the Erie Industrial Park, the nearest industrial facility, use or store materials which are of a nature that could pose a safety hazard to the station due to explosion or fire of any type at this facility. Toxic materials are stored in

limited quantities which do not pose a safety hazard to control room habitability if a release were to occur

2.2.3.3 Aircraft Activities

There are no low-level flight patterns or airport facilities in the proximity of the station site. The existence of the restricted air space in the area of the site reduces the amount of low-level light aircraft operations that could be expected.

2.2.3.4 Use of Restricted Lake Areas

The small arms firing and limited 40 mm anti-aircraft practice firing from Camp Perry has no effect on the safety of the station. The 40mm anti-aircraft training firing was suspended in 1988.

The ordnance test firing from the Erie Industrial Park has no significant effect on the safety of the station due to the type of firing involved, the controls in force in conduct of the firing, and type of ordnance fired. A detailed discussion and analysis of the lake surface restricted areas, airspace restricted areas, and ordnance firing from Camp Perry and the Erie Industrial Park are contained in Appendix 2A.

2.2.3.5 Lake Shipping and Ice Conditions

The distance of the normal shipping lanes from the station site and the distance to deep water available for ships of any size precludes any effect on the station or the station cooling water intake structure from accidents involving ships or barges.

The station cooling water intake structure and arrangement is shown on Figure 2.2-2. The depth of water where this structure is located and low projection above lake bottom precludes any significant damage to the structure from the type of boating using the water areas in the vicinity. Accidental spills of oil or other materials incident with the boating activities can have no effect on the structure or the water drawn into the station through the structure.

Lake Erie is subject to extensive ice formations; however, the depth of the intake structure precludes blockage of intake flow from normal ice formation. The shore area of the station site is subject to ice pileup from northeast wind-driven lake ice. The rockfill barrier beyond the intake structure and the rockfill around the structure itself as shown on Figure 2.2-2 provides protection from ice chunks that could be forced to and along the bottom surface by this piling action. If ice pileup should extend to the lake bottom in the area of the intake structure, it is extremely unlikely that all intake ports could be covered completely and the conservative design of the intake areas provides adequate flow area with extensive partial blockage. The rockfill around the structure has sufficient void areas to permit minimum station flow in the event complete blockage should occur.

In addition to blockage of intake flow by large ice formations, the phenomenon known as frazil ice may also cause blockage of the intake crib at the intake slats. The frazil ice will almost always occur with the combination of temperatures below 20°F, open water above intake crib, clear nights which aid the low temperatures, and strong winds. Small ice crystals are formed and will then accumulate on the intake slats until the ice bridges over the opening and blocks the flow. The intake slats are spaced sufficiently to reduce the probability of ice bridging across the intake openings. Although the frazil ice can not be eliminated, the detrimental affects of the frazil ice on the intake structure are greatly reduced by the large slat spacing.

In any event, the essential safety systems have an adequate supply of impounded cooling water in the intake canal forebay area should all intake flow from the lake be cut off. This is discussed further in Subsection 2.4.11 in consideration of extreme low lake level conditions and in Section 9.2 in discussion of the station cooling water systems.

2.2.3.6 On-Site Facilities

2.2.3.6.1 Water Treatment

Sodium Hydroxide (NaOH), Sodium Hypochlorite (NaOCl) and Sodium Bromide (NaBr) are used at the water treatment facility for the cooling water and cooling tower systems. These chemicals are transported to the site by tank truck and stored in large volume tanks. Accidental release of the algaecide NaOCl and or NaBr would not require evacuation of essential or non-essential personnel from the site. Though the odor may be disagreeable, no toxic threshold limit value (American Conference of Government Industrial Hygienists) or personnel exposure limit (OSHA) has been established for NaOCl or NaBr. The NaOCl tank is a 7500 gallon tank installed within a steel dike capable of containing 110 percent of the tank volume. The NaBr tank is a 4500 gallon tank installed within a concrete dike capable of containing 110 percent of the tank volume. Up to 16,000 gallons of sodium hydroxide are stored at the Water Treatment Plant. Accidental release of the sodium hydroxide would be accumulated in the concrete dike surrounding the tank. Evaluations demonstrate that failure of the sodium hydroxide tank poses no hazard to control room personnel.

A vendor supplied Demineralized Water Treatment package is placed near the Demineralized Water Storage Tank and uses hydrazine and sulfuric acid from approximately 325 gallon portable containers. This amount of chemicals was determined not to be hazardous to the Control Room Operators.

2.2.3.6.2 Fuel Oil Storage

A 100, 000 gallon on-site storage tank for fuel oil for the auxiliary boiler, diesel driven fire pump and miscellaneous diesel generator is located and arranged in such a manner that rupture of this tank with a resulting fire would pose no safety hazard to the station.

Two 40, 000 gallon Emergency Diesel Generator Fuel Oil Storage tanks are installed outdoors, above grade level and are protected by structural backfill. The structural backfill along with vents and flame arresters reduce probability of a fire. The location of the tanks ensure that the effects of a fire would not affect the safe shutdown of the plant.

A 6,000 gallon Emergency Feedwater Facility (EFWF) fuel oil storage tank for supply to the emergency feedwater pump, FLEX “N” pump, and FLEX turbine generator is stored inside of the EFWF on elevation 603’-0”.

A 2,000 gallon Station Blackout Diesel Generator (SBODG) fuel oil storage tank is located within the SBODG building.

2.2.3.6.3 Cooling Tower

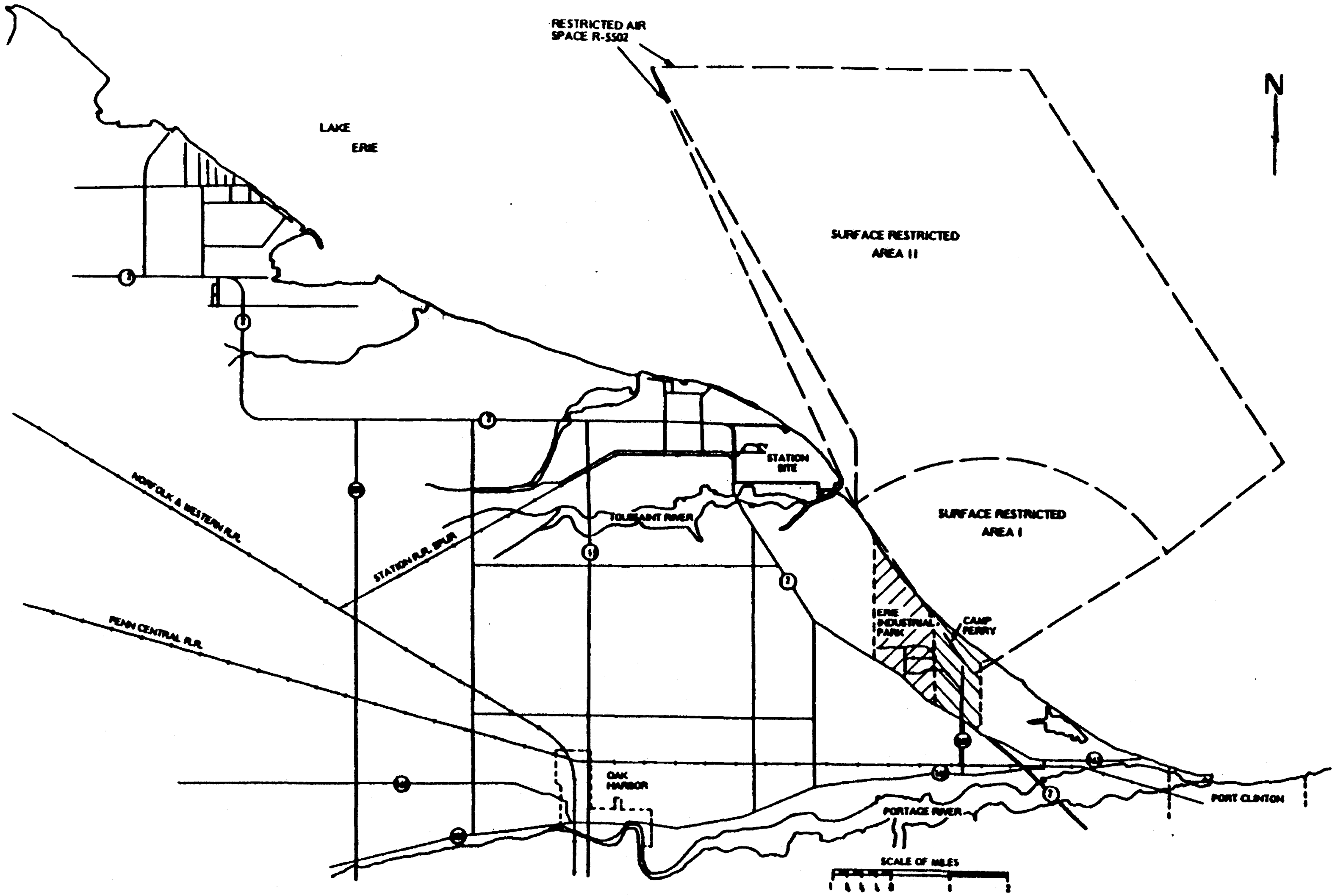
The location of the cooling tower in relation to other station structures is such that complete collapse in the most unfavorable direction would not endanger the critical station structures.

2.2.3.6.4 Secondary System Chemicals

Secondary system chemical additives include Hydrazine, Morpholine/amine compounds, Ammonium Hydroxide, and Hydrogen Peroxide. Evaluations in accordance with Regulatory Guide 1.78 demonstrate that systems use and storage locations do not pose Control Room Habitability (CRH) concerns. New chemical requests and systems modifications affecting CRH are evaluated in accordance with plant administrative procedures.

2.2.3.6.5 Miscellaneous Chemicals

Potentially hazardous chemicals may be stored in quantities greater than the 100 pound criteria of Regulatory Guide 1.78 (December 2001), if the storage quantity and location indicates no affect on Control Room Habitability. Evaluations of onsite chemicals were performed in accordance with Regulatory Guide 1.78 and indicated no Control Room Habitability concerns (Reference 54 and 57). New chemical requests and system modifications affecting Control Room Habitability are evaluated in accordance with plant procedures.



DAVIS-BESSE NUCLEAR POWER STATION
INDUSTRIAL, TRANSPORTATION
AND MILITARY FACILITIES
FIGURE 2.2-1

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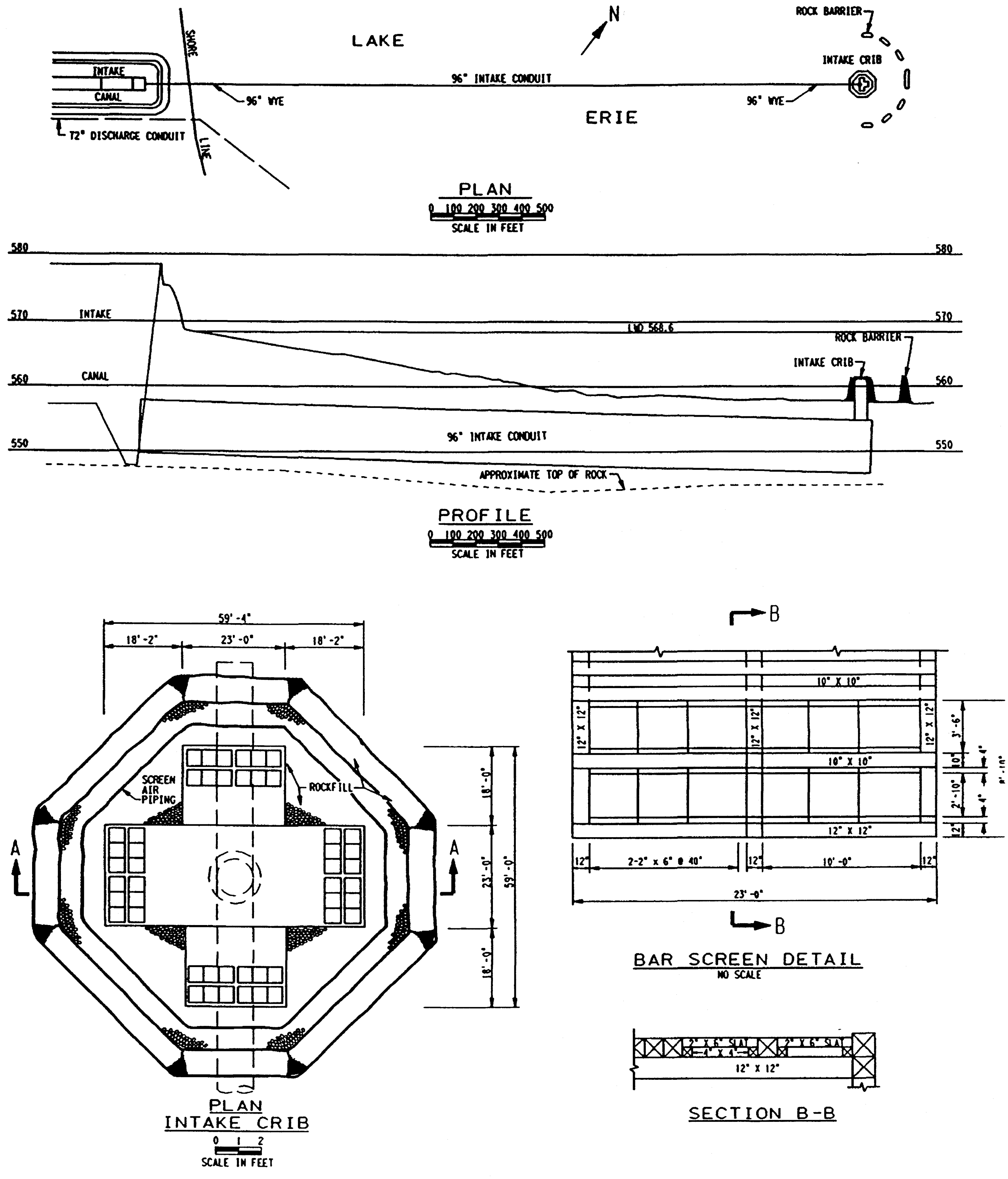

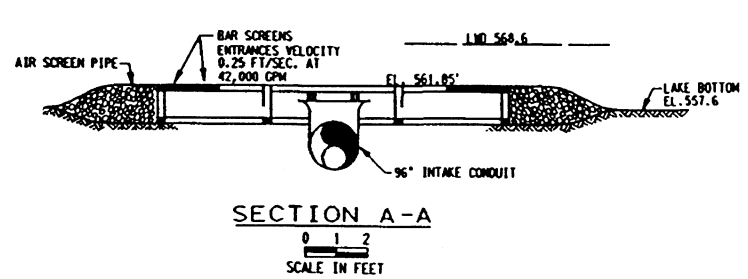

DAVIS-BESSE NUCLEAR POWER STATION

INTAKE ARRANGEMENT

FIGURE 2.2-2

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2.3 METEOROLOGY

2.3.1 Regional Climatology

2.3.1.1 General Climate

The climate of the Davis-Besse Nuclear Power Station can be described as a modified continental type. The proximity to Lake Erie results in less extreme temperatures and more cloudiness and higher humidity than would be experienced farther inland. Precipitation is rather evenly distributed throughout the year and is associated with the frequent passage of cyclonic storms. Five major storm tracks, which originate from Colorado to central Canada, converge on Lake Erie. Cyclonic storms pass over Lake Erie with an average frequency of about one storm every four days.

The station site is located beneath the mean annual position of the belt of strongest westerly winds aloft. As a result, both cyclonic and anti-cyclonic pressure systems tend to move rapidly through the area, and stagnation is rare. In winter the most common source of air masses over the area is from central Canada. This air is colder than the underlying surface and is unstable which favors atmospheric dispersion. In summer the typical air mass has its source from the Gulf of Mexico. Although warm and moist, it is still colder than the underlying land and is unstable. In the fall and early winter as air temperatures drop below the water temperatures of the lakes, instability is induced, and cyclonic storms moving across the Great Lakes tend to intensify. The increased turbulence and higher wind speeds result in more rapid dispersion of atmospheric pollutants.

The land around the site is flat and low lying, being mostly open farmland with occasional small wooded areas. The major local topographical feature likely to affect atmospheric dispersion is Lake Erie. During the spring and summer, when the contrast between the air and water temperatures is the greatest, a “lake breeze” develops, which “bucks” the usual SW gradient wind and leads to a higher frequency of ENE and northerly winds than normally occur in the other three seasons of the year.

Data sources are listed in Section 2.7, References.

2.3.1.2 Severe Weather

2.3.1.2.1 Heavy Precipitation

Precipitation is normally moderate (30-34 inches annually) and fairly evenly distributed throughout the year. Heaviest short-period rainfall is associated with thunderstorms, while longer-period rainfall is associated with migratory frontal systems and cyclonic storms. Additional normal and extreme values of precipitation can be found in Subsection 2.3.2.1.

The maximum 24 hour precipitation by month for Sandusky (72 years of record) are:

Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1.71	2.98	2.96	2.21	3.83	5.95	3.87	4.20	3.29	2.76	2.26	1.74

The maximum recorded point rainfall (inches) for short time intervals for Toledo (1899-1961), Sandusky (1900-1961) and Cleveland (1890-1961) are:

	Minutes					Hours				
	5	10	15	30	60	2	3	6	12	24
Toledo	0.65	1.25	1.78	2.88	3.58	3.65	3.77	4.36	5.88	5.98
Sandusky	0.60	1.20	1.46	2.04	3.51	3.77	4.03	5.10	5.63	5.95
Cleveland	0.78	1.20	1.46	2.09	2.21	3.02	3.29	3.74	4.09	4.97

The return period of extreme short-interval rainfall is shown in Figure 2.3-1. A 6-hour probable maximum precipitation has been calculated based on 10-square mile values, and in the vicinity of the station, the 6-hour value using standard hydrometeorological procedures (ref. 34) is derived at 26.7 inches. This is more than 6 times the amount of rainfall expected at the site once in 100 years and is based on the hydrometeorological procedures in Reference 34 used in performing the analysis in Reference 60.

2.3.1.2.2 Snow

Snowfall is generally moderate (35-40 inches annually), distributed throughout the winter from November through March with frequent thaws. Additional normal and extreme values of snow can be found in subsection 2.3.2.1. The record extremes for Toledo and Cleveland are:

	Max. Monthly Snowfall	Max. 24 Hours
Toledo	26.2 (Jan. 1918)	19.0 (Feb. 1900)
Cleveland	30.5 (Feb. 1908)	17.4 (Nov. 1913)

Snow load data available from an HHFA study conducted in 1952 show that the maximum snowpack of record to be approximately 50 lbs per square foot, the estimated weight of seasonal snowpack equaled or exceeded one year in ten to be 20 lbs. per square foot, and the weight of estimated maximum accumulation on the ground plus weight of maximum probable snowstorm to be 80 lbs. per square foot.

2.3.1.2.3 Ice Storms

Freezing rain can occur in the late fall, winter, and early spring months. During a ten year study by the National Weather Service, the number of days with freezing rain observed at Cleveland were:

Nov	Dec	Jan	Feb	Mar	Apr	Total
2	24	31	34	12	1	104

Accumulations of ice of 0.25 inch once every year and 0.50 inch once every two years can be expected. The mean duration of glaze ice on utility wires if an ice storm occurs (based on an 8 year study of the Edison Electrical Institute) is 34 hours for the State of Ohio as a whole.

2.3.1.2.4 Thunderstorms

Thunderstorms have occurred in the area with the following frequency:

Mean Number of Days per Year with Thunderstorms

Location	Yrs Obs.	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Total
Cleveland	64	0.0	0.0	2.0	2.0	5.0	7.0	7.0	5.0	4.0	2.0	1.0	0.0	35.0
Sandusky	68	0.0	0.0	1.0	2.0	5.0	7.0	7.0	5.0	3.0	1.0	0.0	0.0	31.0
Toledo	10	0.6	0.4	2.8	3.8	5.0	6.9	6.1	5.0	4.0	1.6	0.8	0.5	37.5

2.3.1.2.5 Hail

Hail, according to a study of the Quartermaster Research and Engineering Center, has occurred in the area with the following frequency:

Total Days with Hail

Location	Yrs Obs.	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Total
Cleveland	40	2	2	7	9	6	10	8	6	8	16	4	0	78
Sandusky	40	0	3	8	14	15	13	13	9	2	4	0	0	81
Toledo	40	0	1	5	11	11	11	7	3	0	4	2	0	55

2.3.1.2.6 Tornadoes

Tornadoes are relatively common in Ohio, and the occurrences are geographically evenly distributed throughout the state. Figure 2.3-2 shows the frequency of tornadoes and "tornado days" for Ohio. The monthly distribution (1916-50 and 1955-67) of tornadoes for the State of Ohio is:

Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Total
5	6	24	45	46	42	29	23	15	2	10	0	247

During the same period Ottawa County has experienced one tornado. The mean annual tornado frequency for the period 1953-1962 (ref. 14) was 0.8 for the one-degree square in which the site is located. Using this frequency and method described by Reference 14, the calculated probability of a tornado striking any point within this one-degree square is 6.3×10^{-4} or one tornado every 1,587 years.

2.3.1.2.7 Winds

The highest winds are usually associated with thunderstorms (tornadoes excepted) during the passage of a line squall or cold front. Fastest mile data for Toledo and Cleveland are:

	Toledo (11 Years)		Cleveland (25 Years)	
Month	Speed	Direction	Speed	Direction
Jan	40	W	68	SW
Feb	40	NW	65	W
Mar	56	W	74	W
Apr	72	SW	65	W
May	45	W	68	SW
Jun	49	SW	57	SW
Jul	42	W	65	W
Aug	47	W	61	W
Sep	45	W	45	S
Oct	40	SW	43	W
Nov	65	SW	59	W
Dec	45	SW	49	W

The Task Committee on Wind Forces of the American Society of Civil Engineers has estimated for the area a "fastest mile" of wind, at 30 feet above ground, of approximately 90 mph, once in 100 years.

The extreme fastest mile of wind, at 30 feet above the ground, for the following mean recurrence intervals has been estimated (ref. 14).

Interval (Years)	Annual Extreme-mile (mph)
2	50
10	60
25	80
50	84
100	90

The minimum allowable resultant wind pressure suggested by the National Institute of Standards and Technology (formerly National Bureau of Standards) is:

Elevation (ft)	Wind Pressure (lbs/ft^2)
Less than 30	25
20-49	30
50-99	40
100-499	45

2.3.1.2.8 High Air Pollution Potential

The station site is in an area where periods of high air pollution potential may be expected to occur only 1.0 to 2.0 percent of the time and not persist for long periods. The area is characterized by frequent storm passages, high winds, and thermal instability all of which favor

rapid dispersion of atmospheric pollutants. The climatic data developed and presented for Cleveland and Toledo substantiate this conclusion (ref. 16).

Location	Season	Inversion			Nighttime	
		EST		Total Time	Surface Wind	Cloud Cover
		21	09	(%)	<7 mph	<3/10
Cleveland	Winter				22	22
	Spring				38	30
	Summer				63	54
	Fall				38	42
Toledo	Winter	39	14	23	24	22
	Spring	49	4	22	31	32
	Summer	73	1	30	63	51
	Fall	67	5	36	45	49

Periods of limited dispersion or stagnation often occur in episodes lasting several days. It has been found that such episodes frequently occurred with slow-moving warm-type anticyclones. Reference 24 examined 30 years of United States daily weather maps and tabulated occurrences of anticyclonic conditions, with geostropic wind speeds (wind speeds at about 2000 ft above the ground) less than 17 miles per hour lasting at least 4 days. It found a maximum of over 400 episode-days, about one day in 27, centered over eastern Georgia, tapering off to almost zero just north of the Great Lakes.

The occurrence of limited dispersion episodes throughout the contiguous United States has been objectively determined (ref. 23). The critical limiting conditions used were:

a. All mixing heights 1500 meters or less

b. All mixing layer average wind speeds 4.0 meters per second or less

c. No significant precipitation

d. Above conditions satisfied continuously for at least 2 days

Figure 2.3-3 shows the total number of these episode-days in 5 years to be about 30 episode-days in the vicinity of the station site (ref. 23).

2.3.2 Local Meteorology

2.3.2.1 Normal and Extreme Values of Meteorological Parameters

Resource has been made to the long-term National Weather Service records, primarily from Cleveland and Toledo, to describe those features of the local meteorology which are best described by summaries based on a long period of record. Where differences between Cleveland and Toledo exist, it is likely that Toledo data are more representative of the site than are Cleveland data. Toledo Express airport, where data have been taken since 1955, is about 38 miles west of the station. Cleveland Hopkins International Airport, where data have been

taken since 1956, is about 72 miles to the east-southeast of the station. Both airports are several miles inland from Lake Erie.

2.3.2.1.1 Precipitation

Summaries of total precipitation for Cleveland and Toledo are shown in Tables 2.3-1 and 2.3-2. Snowfall is shown in Table 2.3-3.

2.3.2.1.2 Fog

The mean number of days with heavy fog for Cleveland and Toledo are:

Cleveland (1941-1970), Toledo (1956-1970)

Location	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
Cleveland	2	2	2	1	1	1	1	1	*	1	1	1	13
Toledo	2	2	2	1	1	1	1	2	2	2	2	2	19

*less than one half day per year

2.3.2.1.3 Humidity

Long-term records of relative humidity averaged for 0100, 0700, 1300 and 1900 hours local time for Cleveland and Toledo are:

Cleveland (1960-1970)

Hour	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
0100	73	75	76	72	74	79	80	82	78	75	75	74	76
0700	75	78	78	75	76	78	82	84	83	79	78	75	78
1300	69	69	66	58	57	57	57	59	59	58	66	71	62
1900	71	71	70	61	69	60	62	64	68	67	71	72	66
Average	72	73	72	66	66	68	70	72	72	70	72	73	70

Toledo (1955-1970)

Hour	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
0100	72	72	73	77	77	82	84	86	86	80	81	82	79
0700	79	79	81	81	79	82	86	89	90	84	84	83	83
1300	69	65	62	56	52	54	55	57	56	54	67	73	60
1900	74	70	67	60	57	58	61	65	69	68	74	78	67
Average	74	72	71	68	66	69	72	74	75	72	76	79	72

The annual average relative humidity monitored at the Davis-Besse site at the 35-ft level during the period August 4, 1974, through August 3, 1975, was 73%, which compares favorably with concurrent Toledo data and climatological data from Toledo which both recorded an annual

average of 72% (ref. 4). Table 2.3-4 presents monthly, seasonal, and annual means and extremes of relative and absolute humidity, including mean daily maximum and mean daily minimum values, for Davis-Besse. (The absolute humidity is a measure of the amount of water vapor present to the volume of air occupied by the mixture, usually denoted in grams per cubic meter.)

A more useful design parameter from a cooling tower design standpoint is the wet bulb temperature. Long-term means for Cleveland and Toledo are:

Cleveland (Based on temperature data from 1931 to 1970 and relative humidity data from 1960 to 1970)

Temperature	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
Maximum	31	32	38	49	58	66	69	69	64	54	43	34	69
Mean	25	26	33	42	52	61	66	65	59	49	38	29	45
Minimum	18	19	26	36	46	55	60	59	53	43	32	23	18

Toledo (Based on temperature data from 1931 to 1970 and relative humidity data from 1955 to 1970)

Temperature	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
Maximum	30	31	39	49	58	66	71	70	64	53	43	33	71
Mean	24	25	33	43	53	62	67	65	60	48	37	28	46
Minimum	18	19	27	36	46	56	61	60	53	42	31	22	18

2.3.2.1.4 Temperature

The air temperature of the costal region, being tempered by Lake Erie, is not subject to the wider extremes of the more inland areas. Average and extreme monthly air temperatures (°F) for Cleveland and Toledo are shown in the table below.

Cleveland (1931-1970)

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
Record Max*	68	69	80	85	91	93	98	93	93	86	79	67	98
Mean Max	34	35	43	56	67	76	80	79	73	62	48	37	58
Mean	27	28	36	47	58	68	72	71	65	54	42	31	50
Mean Min	20	20	28	39	50	59	64	42	56	46	35	25	42
Record Min*	-19	-15	5	10	25	39	41	41	34	22	13	-4	-19

*10 years of record (1960 – 1970)
Record max. 103 in July 1941

Toledo (1931-1970)

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
Record Max*	62	68	80	87	95	97	96	98	95	91	78	65	98
Mean Max	33	35	44	57	69	78	83	81	74	62	48	36	58
Mean	26	27	36	48	59	69	73	71	65	53	40	30	50
Mean Min	19	20	28	38	49	59	64	62	55	44	33	23	41
Record Min*	-17	-14	-1	11	26	38	43	37	29	16	2	-11	-17

*15 years of record (1955 – 1970)
Record max. 105 in July 1936

The average ambient temperature recorded onsite during the period August 4, 1974, through August 3, 1975, was 50.5°F, which is in good agreement with long-term averages at Cleveland and Toledo. The monthly, seasonal, and annual mean and extremes of 35-ft. temperature for the Davis-Besse site are presented in Table 2.3-5.

2.3.2.1.5 Diffusion Climatology

Joint frequency distributions of wind speed, wind direction, and stability for the period 1965-1968 were presented in Appendix 2B of Reference 6.

The joint frequency distributions presented in Reference 6 were based on four stability categories designated Moderately Stable, Slightly Stable, Neutral, and Unstable. The relationship between these categories and Pasquill categories, as determined by the vertical temperature gradient (ΔT) is given below:

Reference 6 Category	Pasquill Category	ΔT (°C/100m)
Unstable	A	< - 1.9
	B	-1.9 to -1.7
	C	-1.7 to -1.5
Neutral	D	-1.5 to -0.5
Slightly Stable	E	-0.5 to +1.5
Moderately Stable	F	+1.5 to +4.0
	G	> + 4.0

The vertical temperature difference (ΔT) was used to classify onsite data into atmospheric stability categories. The categories associated with ΔT measurements are shown in Table 2.3-6 (ref. 20). For the Davis-Besse site, the vertical temperature differences between the containment height (approximately 250 feet AGL) and 35 feet AGL were analyzed in detail for the August 4, 1974 through August 3, 1975 period. The annual 35 – foot wind rose is shown by Figure 2.3-4.

Table 2.3-7 presents monthly and annual (ΔT250'-35') stability distribution and associated 35-foot wind speeds for the August 4, 1974 to August 3, 1975 period of record. As can be seen in Table 2.3-7 neutral and slightly stable conditions (D and E) predominate during the data year.

Table 2.3-12 presents monthly and annual onsite wind and stability summaries with stability based on ΔT for the period August 4, 1974 through August 3, 1975.

2.3.2.2 Potential Influence of the Station and It's Facilities on Local Meteorology

The cooling tower and open return channel from the cooling tower to the pump house are the only components of the station that can have any effect on the local meteorology. The environmental effects of the cooling tower have been studied in detail, and this study (Reference 39 which appears as Appendix 7F of the Supplement to the Environmental Report, Davis-Besse Nuclear Power Station) has shown that the effects on local meteorology are negligible.

A visible, elevated plume will form from the cooling tower effluent water vapor under certain conditions. The calculated occurrence of downwash conditions under which the cooling tower effluent is caught in the turbulent wake of the tower structure and brought down to the surface is not a frequent effect, and such downwash occurrences have not been verified in actual cooling tower operations in the United States.

Ground level effects of fog conditions which would be induced by cooling tower operation is negligible since the probability of increased occurrence of fog conditions would be only 0.42% or 3.5 hours at a distance of 25 miles. The probability of increased occurrence of fog is lower at distances closer to the station.

The return canal for cooling water from the cooling tower to the circulation water pump house can produce a small amount of ground fog under certain conditions but this would not extend beyond the station site.

2.3.2.3 Topographical Description

The terrain surrounding the Davis-Besse station is extremely flat. Lake Erie to the north and east has a mean elevation of 571 feet (I.G.L.D). Finished station grade (station structures) elevation is 585 feet (I.G.L.D.). Elevation of all land within 5.0 miles is in the 575 – 590 feet (I.G.L.D.) range (see Figure 2.3-5). Profiles have been provided in the submittal showing compliance with Appendix I, 10CFR50 (ref. 50).

Since only ground level release points are involved, the effect of terrain topography on elevated releases need not be considered.

2.3.3 <u>On-Site Meteorological Measurements Program</u>

The on-site meteorological monitoring system (MMS) complies with the requirements of Regulatory Guide 1.23, "Meteorological Programs in Support of Nuclear Power Plants", proposed Revision 1, dated September 1980.

Meteorological data was originally collected from a 300 foot tower installed by the Research Corporation of New England, a description of which can be found in Reference 4. This tower was replaced because the original system did not comply with Safety Guide 23, "Onsite Meteorological Programs", dated February 17, 1972.

Data collection from a 100 meter (M) tower and a nearly 10M tower began on August 4, 1974. These towers, which are both freestanding, are located within a fenced compound in the southwest corner of the plant. The nearest permanent plant structure is the Warehouse, which is a large, low building 600 feet east-northeast of the towers. In addition, Carroll Township has a 170 foot, 500,000 gallon water tank adjacent to Route 2 about 1200 feet to the northwest of the towers. Excluding transmission towers, the tallest plant structures are the reactor containment building and the cooling tower which are both over 3000 feet to the north. The Lake Erie shoreline is approximately 4000 feet to the northeast at its nearest point.

In November 1990, a backup meteorological system was installed on the two towers. Associated equipment was housed in a new climate controlled shelter located within the fenced compound, and powered from an off-site source. In December 1991, the original system, including its associated equipment in the original shelter, was upgraded to match the configuration of the backup system.

All instruments are mounted on the towers except the precipitation gage. This instrument is mounted on a concrete pad and is surrounded by a drift eliminator. Its location within the fenced compound was selected to avoid window shadow from the predominant wind experienced at the site. The MMS is configured as shown in Table 2.8-8.

Separate primary and backup data are collected and recorded digitally on data loggers located in the two shelters. The data are recorded as 15 minute averages and transmitted to a data acquisition computer in the Technical Support Center. Information supplied by the computer includes the following:

- 15/60 minute average wind speed/direction
- Wind direction standard deviation
- 15/60 minute average temperature, dew point, and differential temperature
- Pasquill stability class based on 75M minus 10M differential temperature
- Hourly joint frequency distribution of wind speed versus direction by stability class

This information is available to Operators in the Control Room.

The data processing system is capable of making automatic adjustments to compensate for instrument drift and identifying instruments with excessive drift. All instruments (except chart recorders) are calibrated at least once every six months in accordance with written procedures and the Technical Requirements Manual (TRM). The TRM is incorporated by reference into the Davis-Besse USAR.

Joint frequency distribution of wind speed versus wind direction by delta-T stability class is available to the unit operator at any time.

Monthly and annual onsite wind stability summaries with stability based on delta-T for the period of August 4, 1974, through August 3, 1975, are presented in Table 2.3-12.

2.3.4 Short-Term (Accident) Diffusion Estimates

2.3.4.1 Objective

Onsite data from Davis-Besse for August 4, 1974 through August 3, 1975 have been used to evaluate hypothetical accident meteorology. This hypothetical accident is postulated to characterize upper limit concentrations and dosages that might occur in the event of a containment release. Among the basic inputs to the accident analysis are the meteorological conditions which determine the dilution capacity of the atmosphere. The meteorological conditions postulated for the hypothetical accident are based on analysis of available Davis-Besse site meteorological data.

An additional year of onsite meteorological data for the period of August 1975 to August 1976 was submitted to the NRC. These data were used by the NRC to re-evaluate the relative concentration values (X/Q) for the new year of data, as well as the combined data for August 1974 through August 1976. The NRC found that the original data representing August 1974 through August 1975, which are included in this chapter, are representative and conservative.

2.3.4.2 Calculations

Cumulative frequency distributions of X/Q values have been obtained for the following interval lengths: 1 hour, 2 hours, 8 hours, 16 hours, 72 hours (3 days), and 624 hours (26 days). Only centerline X/Q values were used for interval lengths of 1, 2, and 8 hours. Only sector average X/Q values were used for interval lengths of 16, 72, and 624 hours.

Centerline X/Q values were calculated using the following formula (refs. 43 and 44):

$$\frac{1}{X/Q = \left(\pi\sigma_y\sigma_z + cA\right)\bar{u}}$$

where:

X/Q = the calculated atmospheric dispersion parameter, s/m3

σ_y and σz = horizontal and vertical dispersion parameters, m

$\bar{u}$ = the mean wind speed, m/sec

c = the building shape factor (0.5), dimensionless

A = the minimum cross-sectional area of the Containment building
= [(height x diameter) + dome cross-sectional area.] x 0.0929 m^2/ft^2
= [(217.25 ft. x 144 ft.) + 2398.9 ft2] x 0.0929 m^2/ft^2
= 3129 m^2

Inclusion of the building wake factor (cA) was not allowed to reduce any calculated centerline X/Q by more than a factor of three.

Sector average X/Q values were calculated using the following relationship (ref. 45):

$$X/Q = \frac{2.032}{\sigma_z ux}$$

Where:

x = the downwind distance, m.

Wind data at a height of 35 ft. above ground were used in the calculations. Vertical temperature lapse rates measured at $\Delta T_{250' - 35'}$ were used to obtain both vertical and horizontal dispersion parameters. Calms were assigned the wind direction of the first following non-calm observation and a wind speed of 0.25 mph. The set of valid hourly observations was collapsed around observations of no data, and a consecutive set of 8155 valid hourly observations, out of a possible total of 8760, was obtained. These data were linked, end to beginning, to form an endless cycle.

Cumulative frequency distributions of X/Q values for interval lengths of 1 and 2 hours were obtained for the boundary of the site exclusion area, which, for Davis-Besse, is the same as the site boundary. Since there is nothing but water beyond the site boundary to the northeast, east-northeast, and east, it is highly unlikely that a receptor individual will be located in these directions should an accident occur. For this reason, all winds blowing toward these directions were assumed to result in zero values of X/Q at the exclusion area boundary. Cumulative frequency distributions of X/Q values for interval lengths of 8, 16, 72, and 624 hrs, were obtained for the outer boundary of the Low Population Zone (LPZ). For Davis-Besse, the low-population zone extends outward from the facility to a distance of 3200m in all directions. There is only water beyond the LPZ boundary to the north-northwest, north, north-northeast, northeast, east-northeast, and east. Since it is highly unlikely that a receptor individual will be present in any of these directions during the course of an accident, all winds blowing toward these directions were assumed to result in zero values of X/Q at the LPZ boundary.

Cumulative frequency distributions of X/Q values were obtained by using the following procedure for each interval length:

a. The hourly X/Q values were computed at the required distances for each hourly observation (either all centerline or all sector average).

b. The interval was allowed to begin at each of the 8155 hourly observations for which valid data were available.

c. For each interval start time of 8155 possible start times, the interval average value of X/Q in each of the 16 directions was computed and the maximum interval average X/Q was stored in a master file.

d. After obtaining a complete set of maximum interval average values of X/Q (one per possible start time, 8155 values altogether) the calculated values were ranked from highest to lowest and the required percentile values were selected.

The results of these analyses are presented in Table 2.3-10.

The actual site boundary distances used in the analysis are:

Direction	Site Boundary Distance (meters)
NNE	730
NE	825
ENE	960
E	1210
ESE	1565
SE	1075
SSE	915
S	880
SSW	900
SW	965
WSW	830
W	815
WNW	835
NW	870
NNW	740
N	720

2.3.5 Long-Term (Routine) Diffusion Estimates

Annual average atmospheric dilution factors (X/Q) were calculated for Davis-Besse based on site data for the August 4, 1974, through August 3, 1975, period and using the following equation, which assumes a uniform horizontal distribution within a 22.5 degree sector and a ground level release (References 48 and 49):

$$\left[\frac{X}{Q}\right]_j = \left[\frac{2.032}{x}\right]\sum_{i=1}^{m}\frac{F_{ij}}{S_{z_i}}\left\langle\frac{1}{u_{ij}}\right\rangle_{ave}$$

Where:

$(\phi/Q)_j$ = relative ground level concentration (Φ) normalized to source strength (Q) for section j, seconds per cubic meter

S_{z_i} = effective vertical dispersion parameter for stability class i, meters

$\left\langle\frac{1}{u_{ij}}\right\rangle ave$ = average inverse wind speed for stability class i and sector j, seconds per meter

F_{ij} = fraction of time (based on all observations) that class i occurs within sector j

x = downwind distance, meters

m = number of stability categories, seven (A through G).

An effective σ_z parameter, S_{z_i} , is used to account for building wake effects as follows (refs. 21 and 22):

$$s_{z_i} = \left[\sigma_{z_i}^2 + \frac{cH^2}{\pi} \right]^{1/2}$$

With the constraint that (Ref. 20)

$$s_{z_i} \leq \sqrt{3}\sigma_{z_i}$$

Where

σ_{z_i}	=	vertical stability parameter for stability class i, meters
c	=	building shape factor (0.5), dimensionless
H	=	the height of the reactor building = 73.5 m.

Stability was based on $\Delta T_{250'-35'}$ data. Wind data used were from the 35 ft. level. Calms were distributed among the sectors in proportion to the distribution of winds in the 0.6 to 1.5 mph range and were assigned a speed of 0.25 mph.

Table 2.3-11 presents annual average site boundary X/Q values for the August 4, 1974 to August 3, 1975 period. Table 2.3-12 presents annual average X/Q values for the 16 radial sectors for incremental distances to 50 miles.

TABLE 2.3-1

Total Precipitation Cleveland

Month	Mean Days with 0.01 Inch or More (1)	Mean (inches) (2)	Extreme Monthly Maximum (inches) (3)	Extreme Monthly Minimum (inches) (3)	Max. In 24 hours (inches) (2)
Dec	16	2.38	5.60	0.71	1.89
Jan	16	2.55	7.01	0.36	2.33
Feb	14	2.33	4.64	0.73	3.62
Winter	46	7.26			
Mar	16	2.84	6.07	0.78	3.17
Apr	15	2.80	6.61	1.13	2.24
May	13	3.12	6.04	0.58	3.73
Spring	44	8.76			
Jun	11	3.22	9.77 (1902)	0.39	3.10
Jul	10	3.40	6.47	0.96	3.86
Aug	9	2.89	6.95	0.17 (1881)	4.13
Summer	30	9.51			
Sep	9	3.07	6.37	0.74	4.97
Oct	10	2.61	9.50	0.61	3.44
Nov	15	2.62	6.44	0.92	2.23
Fall	34	8.30			
Annual	154	33.83			

(1) For Period 1931 – 1970 Inclusive
(2) For Period 1871 – 1970 Inclusive
(3) For Period 1931 – 1970 Inclusive (unless otherwise indicated)

All data are from 30 years of record 1931 – 1970 unless otherwise indicated.

TABLE 2.3-2

Total Precipitation Toledo

Month	Mean Days with 0.01Inch or More (1)	Mean (Inches) (2)	Extreme Monthly Maximum (Inches) (3)	Extreme Monthly Minimum (Inches) (3)	Max. In 24 hours (Inches) (2)
Dec	14	2.32	6.81	0.54	3.53
Jan	13	2.21	6.30	0.27	1.78
Feb	11	1.92	4.32	0.27	2.26
Winter	38	6.45			
Mar	14	2.56	5.22	0.58	2.69
Apr	13	2.82	5.97	0.84	2.93
May	12	3.21	8.04	0.69	3.57
Spring	39	8.55			
Jun	10	3.39	6.67	0.12	3.44
Jul	10	3.00	6.75	1.17	4.39
Aug	8	2.86	8.47	0.81	4.58
Summer	28	9.25			
Sep	10	2.61	6.70	0.37	5.98
Oct	8	2.30	8.49 (1881)	0.28	3.10
Nov	11	2.33	4.63	0.04 (1904)	2.68
Fall	29	7.24			
Annual	133	31.53			

(1) For Period 1956 – 1970 Inclusive
(2) For Period 1871 – 1970 Inclusive
(3) For Period 1931 – 1970 Inclusive (unless otherwise indicated)

TABLE 2.3-3

Average Snowfall Data for Cleveland and Toledo

Cleveland (1941 – 1970)

Snowfall	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sept	Oct	Nov	Dec	Annual
Avg Monthly	10.1	10.4	10.3	2.1	T	0.0	0.0	0.0	T	0.6	5.6	10.7	49.8
Max. Monthly *	18.7	20.7	26.3	14.5	0.1	0.0	0.0	0.0	T	8.0	22.3	30.3	-
Min. Monthly *	0.5	2.0	1.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	T	T	-
Max. 24 Hrs	9.3	7.5	14.9	7.6	0.1	0.0	0.0	0.0	T	6.7	15.0	9.3	-
Avg No. Days ≥ Inch	4	4	3	1	0	0	0	0	0	*	2	4	18

* 1931-32 to 1970 Inclusive
T – Trace and amount too small to measure
Max. Monthly 30.5, February 1908
Max. In 24 hours 17.4 November 1913

Toledo (1956 – 1970)

Snowfall	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sept	Oct	Nov	Dec	Annual
Avg Monthly	8.8	7.8	7.2	2.1	T	0.0	0.0	0.0	T	T	3.4	8.1	37.4
Max. Monthly *	16.4	14.4	14.6	12.0	T	0.0	0.0	0.0	T	0.5	17.9	25.5	-
Min. Monthly *	0.3	0.6	T	0.0	0.0	0.0	0.0	0.0	0.0	0.0	T	T	-
Max. 24 Hours	6.6	7.4	7.5	9.8	T	0.0	0.0	0.0	T	0.2	8.3	6.1	-
Avg No. Days ≥ 1.0 Inch	3	2	2	1	0	0	0	0	0	0	1	2	11

* 1931-32 to 1970 Inclusive
T – Trace and amount too small to measure
Max. monthly 26.2 January 1918
Max. in 24 hours 19.0 February 1900

TABLE 2.3-4

Monthly, Seasonal and Annual Means and Extremes of Relative

and Absolute Humidity (35 ft) at the Davis-Besse Site

(August 4, 1974 – August 3, 1975)

Relative (%) and Absolute * (g/m^3) Humidity	AUG	SEP	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	SPR	SUM	FALL	WIN	ANNUAL
Mean Daily Max	88 (15)	87 (10)	78 (7)	89 (6)	84 (3)	81 (3)	75 (2)	79 (3)	73 (4)	80 (10)	85 (14)	81 (13)	76 (6)	85 (14)	84 (7)	80 (3)	81 (8)
Mean Daily Min	66 (14)	57 (9)	54 (6)	76 (5)	79 (3)	70 (3)	68 (2)	65 (3)	63 (4)	57 (9)	65 (13)	54 (12)	60 (6)	62 (13)	62 (7)	73 (3)	64 (7)
Highest	100 (20)	100 (19)	100 (15)	100 (15)	100 (6)	100 (16)	100 (9)	100 (11)	100 (13)	100 (19)	100 (21)	100 (20)	100 (19)	100 (21)	100 (19)	100 (16)	100 (21)
Lowest	39 (8)	29 (3)	22 (3)	50 (1)	56 (1)	40 (1)	37 (0)	22 (1)	34 (1)	21 (3)	42 (7)	38 (7)	21 (1)	38 (7)	22 (1)	37 (0)	21 (0)
Monthly Mean	77 (14)	73 (10)	67 (6)	84 (6)	82 (3)	76 (3)	71 (2)	72 (3)	67 (4)	69 (10)	75 (13)	66 (13)	69 (6)	73 (13)	75 (7)	77 (3)	73 (7)

* Absolute humidity in parentheses

TABLE 2.3-5

Monthly and Annual Means and Extremes of 35-ft Temperature
For the Davis-Besse Site
(August 4, 1974 – August 3, 1975)

35 – ft TEMPERATURE (ºF)	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
Mean Daily Max	35.8 (31)	33.3 (27)	38.9 (29)	47.0 (30)	70.3 (31)	75.5 (30)	80.1 (31)	77.8 (31)	68.6 (30)	59.6 (31)	47.9 (30)	34.0 (31)	56.0 (362)
Mean Daily Min	25.1 (31)	24.4 (27)	28.9 (29)	36.8 (30)	57.6 (31)	64.3 (30)	65.9 (31)	66.3 (31)	54.8 (30)	44.8 (31)	37.1 (30)	26.9 (31)	44.7 (362)
(Min + Max)/2	30.4	28.9	33.9	41.9	64.0	69.9	73.0	72.0	61.7	52.2	42.5	30.5	50.3
Highest Date	59.5 10/75	52.0 22/75	65.0 24/75	74.5 30/75	88.0 25/75	90.0 22/75	87.0 23/75	86.0 1/75	82.0 11/74	78.0 6/74	72.5 1/74	42.0 3/74	90.0
Lowest Date	10.5 13/75	-5.5 10/75	18.5 9/75	22.5 5/75	46.5 2/75	51.5 3/75	55.0 14/75	57.5 5/75	41.0 23/74	28.5 21/74	20.5 26/74	14.5 5/74	-5.5
Monthly Mean	30.7 (734)	28.4 (594)	33.4 (647)	41.4 (710)	63.8 (739)	69.7 (709)	73.6 (712)	72.2 (718)	61.3 (713)	52.2 (732)	43.3 (679)	30.7 (729)	50.5 (8416)

Note: Number of observations is given in parentheses.

TABLE 2.3-6

Vertical ΔT Stability Categories

Pasquill Stability Class			Range of Vertical Temperature Gradient (ºC/100m)					Turbulence Type
A	=	Very Unstable			ΔT	<	-1.9	High Atmospheric Turbulence
B	=	Moderately Unstable	-1.9	≤	ΔT	<	-1.7	
C	=	Slightly Unstable	-1.7	≤	ΔT	<	-1.5	
D	=	Neutral	-1.5	≤	ΔT	<	-0.5	Moderate Atmospheric Turbulence
E	=	Slightly Stable	-0.5	≤	ΔT	<	1.5	Low Atmospheric Turbulence
F	=	Moderately Stable	1.5	≤	ΔT	<	4.0	
G	=	Very Stable	4.0	≤	ΔT			

TABLE 2.3-7

Stability Distributions Based on $\Delta T_{250' - 35'}$ with
Associated Average Wind Speed (35 ft) Data for the Davis-Besse Site
(August 4, 1974 – August 3, 1975)

	Stability Category						
	A	B	C	D	E	F	G
August							
%	0.86	2.01	7.76	50.43	25.72	10.20	3.02
mph	7.9	10.8	9.7	8.1	5.6	3.5	3.4
September							
%	3.80	6.57	8.03	50.66	18.83	8.76	3.36
mph	9.6	12.4	11.1	10.4	7.1	4.2	4.6
October							
%	1.23	6.01	6.83	44.81	25.00	8.61	7.51
mph	10.3	12.8	11.3	10.2	7.6	5.2	5.0
November							
%	2.66	2.37	3.70	59.91	21.45	7.40	2.51
mph	7.1	13.0	12.1	13.1	7.9	6.1	4.2
December							
%	0.00	0.16	0.16	43.09	43.26	10.86	2.47
mph	--	8.5	8.0	11.3	8.0	6.1	3.8
January							
%	0.00	0.00	0.00	58.00	38.17	3.69	0.14
mph	--	--	--	13.9	12.0	7.2	11.5
February							
%	0.00	0.17	0.17	63.30	30.81	4.04	1.52
mph	--	14.0	15.0	13.7	11.3	7.1	6.4
March							
%	0.00	0.00	0.48	64.76	28.10	4.29	2.38
mph	--	--	21.7	15.1	10.7	8.1	7.5
April							
%	0.00	0.15	0.75	74.78	17.01	4.93	2.39
mph	--	24.0	22.8	13.1	10.5	7.1	6.7
May							
%	0.54	2.17	4.74	50.41	31.84	7.59	2.71
mph	14.7	12.2	10.8	9.5	7.1	6.2	5.7
June							
%	2.02	3.47	6.50	54.34	31.07	2.02	0.58
mph	13.2	11.2	12.7	10.5	8.1	5.8	3.9

TABLE 2.3-7 (Continued)

Stability Distributions Based on $\Delta T_{250' - 35'}$ with
Associated Average Wind Speed (35 ft) Data for the Davis-Besse Site
(August 4, 1974 – August 3, 1975)

	Stability Category						
	A	B	C	D	E	F	G
July							
%	2.27	10.50	9.93	31.35	25.11	14.47	6.38
mph	7.0	8.1	7.9	9.6	6.8	4.8	3.8
Annual							
%	1.14	2.89	4.22	53.59	27.94	7.27	2.95
mph	9.4	11.0	10.8	11.8	8.6	5.5	4.9

TABLE 2.3-8

Meteorological Sensor Locations and Specifications

Parameter	Location of Sensor (M) GL	10	75	100	Performance Criteria
Wind Speed		X	X	X	± 0.22 m/s (0.5 mph) for speeds less than 11.13 m/s (25 mph), with a starting threshold of less than 0.45 m/s (1 mph).
Wind Direction		X	X	X	± 5º of azimuth, with a starting threshold of less than 0.45 m/s (1 mph). If the sensor is to be used for standard deviation data, the damping ratio must be 0.4 to 0.6, inclusive, with a deflection of 15º and delay distance not to exceed 2 meters.
Temperature		X			± 0.5º C (0.9º F)
Differential Temperature (DT)		X-- X--	---X -----	 ---X	± 0.15º C (0.27º F) per 50-meter height interval.
Dew Point		X			± 1.5º C (2.7º F) or an equivalent accuracy for relative humidity or wet bulb temperature. These accuracies are applicable for conditions where relative humidity is in excess of 60 percent and temperature is between -30º and 30º C (-22º and 86º F), which is the region of concern for evaluation.
Precipitation	X				By a recording rain gauge with a resolution of 0.25 mm (0.01 in). The accuracy of the recorded value must be within ±10 percent of the total accumulated catch.

TABLE 2.3-10

Short-Term (Accident) X/Q Values for the Davis-Besse Site
(August 4, 1974 – August 3, 1975)

Accident Period (hr)	Distance	X/Q (sec/m³) 5% Probability Value	50% Probability Value	Worst Case
1	Actual site boundary	1.9×10^{-4}	2.5×10^{-5}	--
2	Actual site boundary	1.5×10^{-4}	2.2×10^{-5}	--
8	LPZ (3200 m)	9.9×10^{-6}	1.7×10^{-6}	1.2×10^{-4}
16	LPZ	2.3×10^{-6}	6.5×10^{-7}	1.9×10^{-5}
72 (3 days)	LPZ	1.5×10^{-6}	4.5×10^{-7}	4.2×10^{-6}
624 (26 days)	LPZ	5.6×10^{-7}	3.5×10^{-7}	6.0×10^{-7}

TABLE 2.3-11

Annual Average X/Q at the Site Boundary
(August 4, 1974 – August 3, 1975)

Exposure Direction	Wind Direction	Site Boundary Distance (m)	Site Boundary X/Q (sec/m³) 8/4/74 – 8/3/75
NNE	SSW	730	3.8×10^{-6}
NE	SW	825	3.1×10^{-6}
ENE	WSW	960	1.6×10^{-6}
E	W	1210	9.0×10^{-7}
ESE	WNW	1565	2.6×10^{-7}
SE	NW	1075	5.5×10^{-7}
SSE	NNW	915	5.5×10^{-7}
S	N	880	5.2×10^{-7}
SSW	NNE	900	6.0×10^{-7}
SW	NE	965	8.4×10^{-7}
WSW	ENE	830	1.0×10^{-6}
W	E	815	1.5×10^{-6}
WNW	ESE	835	1.0×10^{-6}
NW	SE	870	1.1×10^{-6}
NNW	SSE	740	1.9×10^{-6}
N	S	720	3.5×10^{-6}

TABLE 2.3-12

Onsite Wind and Stability Summaries with Stability Based on ΔT

WIND SPEED GROUP DESCRIPTION

Group	Windspeed Range (MPH)
1	Less than 0.5
2	0.50 – 3.5
3	3.51 – 7.5
4	7.51 – 12.5
5	12.51 – 18.5
6	18.51 – 24.0
7	> 24.0

WIND SPEED GROUP EQUIVALENTS

(MPH)	Calm	1-3	4-7	8-12	13-18	18-24	GT 24
MPH	LT 0.5	0.5 - 3.5	3.6 - 7.5	7.6 - 12.5	12.6 - 18.5	18.6 - 24.0	GT 24.0
MPS	LT 0.2	0.2 - 1.5	1.6 - 3.3	3.4 - 5.6	5.7 - 8.3	8.4 - 10.9	GT 11.0
KTS	LT 0.4	0.4 - 3.0	3.1 - 6.5	6.6 - 10.8	10.9 - 16.1	16.2 - 21.3	GT 21.4

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

24–HOUR SUMMARY OF WIND SPEED DISTRIBUTION

Toledo Edison 35–FT Winds ($\Delta T_{250'-35'}$) AUG 1974 and 1975

TOTAL NUMBER OF READINGS	6.96000E+02
TOTAL NUMBER OF READINGS WITHOUT CALMS	6.94000E+02

****

WIND SPEED DISTRIBUTION, NO OF OBS.

LT .5	.50 – 3.5	3.51 – 7.5	7.51 – 12.5	12.51 – 18.5	18.51 – 24.0	GT 24.0
2	135	284	238	31	5	1

****

SUMMED OVER ALL DIRECTIONS
WIND SPEED DISTRIBUTION VERSUS TEMP. LAPSE RATE STABILITY CLASS (NO OF OBS.)

	A	B	C	D	E	F	G
1	0	0	0	0	2	0	0
2	2	0	1	32	44	44	12
3	1	3	13	139	93	26	9
4	0	5	35	157	40	1	0
5	3	6	3	19	0	0	0
6	0	0	2	3	0	0	0
7	0	0	0	1	0	0	0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO EDISON 35–FT WINDS ($\Delta T_{250'-35'}$) AUG 1974 and 1975

****

SUMMED OVER ALL TEMP. LAPSE RATE STABILITIES
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND DIRECTION	WIND SPEED (MPH) AT 10–METER LEVEL 1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	1	10	2	1	0	0	14
NE	2	16	38	3	0	0	59
ENE	2	25	29	2	0	0	58
E	6	37	23	0	0	0	66
ESE	10	19	8	0	0	0	37
SE	10	23	13	0	0	0	46
SSE	21	18	7	0	0	0	46
S	30	20	8	0	0	0	58
SSW	11	52	25	0	0	0	88
SW	21	22	27	13	3	0	86
WSW	7	12	10	10	2	0	41
W	4	9	12	2	0	0	27
WNW	4	7	12	0	0	0	23
NW	5	5	14	0	0	1	25
NNW	1	4	4	0	0	0	9
N	0	5	6	0	0	0	11
TOTAL	135	284	238	31	5	1	694

PERIODS OF CALM (NO. OF HOURS) – 2

MISSING DATA (NO. OF HOURS) – 48

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO EDISON 35–FT WINDS ($\Delta T_{250'-35'}$) AUG 1974 and 1975

TEMP. LAPSE RATE STABILITY CLASS A
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10-METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	0	0	0	0	0	0
NE	0	0	0	0	0	0	0
ENE	0	0	0	1	0	0	1
E	0	0	0	0	0	0	0
ESE	1	0	0	0	0	0	1
SE	0	0	0	0	0	0	0
SSE	0	0	0	0	0	0	0
S	0	1	0	0	0	0	1
SSW	0	0	0	0	0	0	0
SW	1	0	0	2	0	0	3
WSW	0	0	0	0	0	0	0
W	0	0	0	0	0	0	0
WNW	0	0	0	0	0	0	0
NW	0	0	0	0	0	0	0
NNW	0	0	0	0	0	0	0
N	0	0	0	0	0	0	0
TOTAL	2	1	0	3	0	0	6

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (CONTINUED)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO EDISON 35–FT WINDS ($\Delta T_{250'-35'}$) AUG 1974 and 1975

TEMP. LAPSE RATE STABILITY CLASS B
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND DIRECTION	WIND SPEED (MPH) AT 10-METER LEVEL 1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	1	0	0	0	0	1
NE	0	0	0	0	0	0	0
ENE	0	0	1	1	0	0	2
E	0	1	0	0	0	0	1
ESE	0	0	0	0	0	0	0
SE	0	0	0	0	0	0	0
SSE	0	0	0	0	0	0	0
S	0	1	0	0	0	0	1
SSW	0	0	0	0	0	0	0
SW	0	0	0	2	0	0	2
WSW	0	0	0	3	0	0	3
W	0	0	0	0	0	0	0
WNW	0	0	2	0	0	0	2
NW	0	0	2	0	0	0	2
NNW	0	0	0	0	0	0	0
N	0	0	0	0	0	0	0
TOTAL	0	3	5	6	0	0	14

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO EDISON 35–FT WINDS ($\Delta T_{250'-35'}$) AUG 1974 and 1975

TEMP. LAPSE RATE STABILITY CLASS C
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10-METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	2	0	0	0	0	2
NE	0	0	4	0	0	0	4
ENE	0	1	10	0	0	0	11
E	0	3	4	0	0	0	7
ESE	0	1	0	0	0	0	1
SE	0	1	0	0	0	0	1
SSE	0	0	0	0	0	0	0
S	0	0	0	0	0	0	0
SSW	0	4	4	0	0	0	8
SW	1	1	4	0	1	0	7
WSW	0	0	1	2	1	0	4
W	0	0	2	1	0	0	3
WNW	0	0	3	0	0	0	3
NW	0	0	2	0	0	0	2
NNW	0	0	1	0	0	0	1
N	0	0	0	0	0	0	0
TOTAL	1	13	35	3	2	0	54

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO EDISON 35–FT WINDS ($\Delta T_{250'-35'}$) AUG 1974 and 1975

TEMP. LAPSE RATE STABILITY CLASS D
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND DIRECTION	WIND SPEED (MPH) AT 10-METER LEVEL 1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	1	7	2	1	0	0	11
NE	0	15	34	3	0	0	52
ENE	1	18	15	0	0	0	34
E	3	26	16	0	0	0	45
ESE	6	16	8	0	0	0	30
SE	1	4	10	0	0	0	15
SSE	2	4	7	0	0	0	13
S	5	6	5	0	0	0	16
SSW	2	16	12	0	0	0	30
SW	4	11	15	9	2	0	41
WSW	1	0	7	5	1	0	14
W	0	3	6	1	0	0	10
WNW	2	3	5	0	0	0	10
NW	3	3	7	0	0	1	14
NNW	1	2	4	0	0	0	7
N	0	5	4	0	0	0	9
TOTAL	32	139	157	19	3	1	351

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO EDISON 35–FT WINDS ($\Delta T_{250'-35'}$) AUG 1974 and 1975

TEMP. LAPSE RATE STABILITY CLASS E
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND DIRECTION	WIND SPEED (MPH) AT 10-METER LEVEL 1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	0	0	0	0	0	0
NE	2	1	0	0	0	0	3
ENE	0	6	3	0	0	0	9
E	3	7	3	0	0	0	13
ESE	3	2	0	0	0	0	5
SE	4	17	3	0	0	0	24
SSE	7	8	0	0	0	0	15
S	13	5	3	0	0	0	21
SSW	3	19	8	0	0	0	30
SW	4	6	8	0	0	0	18
WSW	1	11	2	0	0	0	14
W	0	4	4	0	0	0	8
WNW	2	3	2	0	0	0	7
NW	2	2	3	0	0	0	7
NNW	0	2	0	0	0	0	2
N	0	0	1	0	0	0	1
TOTAL	44	93	40	0	0	0	177

PERIODS OF CALM (NO. OF HOURS) – 2

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO EDISON 35–FT WINDS ($\Delta T_{250'-35'}$) AUG 1974 and 1975

TEMP. LAPSE RATE STABILITY CLASS F
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10-METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	0	0	0	0	0	0
NE	0	0	0	0	0	0	0
ENE	1	0	0	0	0	0	1
E	0	0	0	0	0	0	0
ESE	0	0	0	0	0	0	0
SE	5	1	0	0	0	0	6
SSE	10	5	0	0	0	0	15
S	9	3	0	0	0	0	12
SSW	6	10	1	0	0	0	17
SW	9	3	0	0	0	0	12
WSW	2	1	0	0	0	0	3
W	2	2	0	0	0	0	4
WNW	0	1	0	0	0	0	1
NW	0	0	0	0	0	0	0
NNW	0	0	0	0	0	0	0
N	0	0	0	0	0	0	0
TOTAL	44	26	1	0	0	0	71

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO EDISON 35–FT WINDS ($\Delta T_{250'-35'}$) AUG 1974 and 1975

TEMP. LAPSE RATE STABILITY CLASS G
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND DIRECTION	WIND SPEED (MPH) AT 10-METER LEVEL 1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	0	0	0	0	0	0
NE	0	0	0	0	0	0	0
ENE	0	0	0	0	0	0	0
E	0	0	0	0	0	0	0
ESE	0	0	0	0	0	0	0
SE	0	0	0	0	0	0	0
SSE	2	1	0	0	0	0	3
S	3	4	0	0	0	0	7
SSW	0	3	0	0	0	0	3
SW	2	1	0	0	0	0	3
WSW	3	0	0	0	0	0	3
W	2	0	0	0	0	0	2
WNW	0	0	0	0	0	0	0
NW	0	0	0	0	0	0	0
NNW	0	0	0	0	0	0	0
N	0	0	0	0	0	0	0
TOTAL	12	9	0	0	0	0	21

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

24 HOUR SUMMARY OF WIND SPEED DISTRIBUTION

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) SEPT 1974

TOTAL NUMBER OF READINGS 6.85000E+02

TOTAL NUMBER OF READINGS WITHOUT CALMS 6.84000E+02

****

WIND SPEED DISTRIBUTION, NO OF OBS.

LT .5	.50 – 3.5	3.51 – 7.5	7.51 – 12.5	12.51 – 18.5	18.51 – 24.0	GT 24.0
1	51	257	245	96	35	0

****

SUMMED OVER ALL DIRECTIONS
WIND SPEED DISTRIBUTION VERSUS TEMP. LAPSE RATE STABILITY CLASS (NO OF OBS.)

	A	B	C	D	E	F	G
1	0	0	0	0	1	0	0
2	0	1	1	11	9	22	7
3	13	7	8	103	73	38	15
4	6	17	30	150	41	0	1
5	6	13	12	60	5	0	0
6	1	7	4	23	0	0	0
7	0	0	0	0	0	0	0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) SEPT 1974

****

SUMMED OVER ALL TEMP. LAPSE RATE STABILITIES
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND DIRECTION	WIND SPEED (MPH) AT 10 METER LEVEL 1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	2	22	26	14	10	0	74
NE	2	11	22	7	3	0	45
ENE	3	8	19	2	0	0	32
E	2	9	10	0	0	0	21
ESE	0	5	1	0	0	0	6
SE	0	9	2	0	0	0	11
SSE	2	21	6	0	0	0	29
S	12	46	15	0	0	0	73
SSW	7	58	45	3	0	0	113
SW	13	36	42	18	5	0	114
WSW	4	9	8	9	2	0	32
W	2	6	14	9	4	0	35
WNW	1	3	7	12	10	0	33
NW	0	1	8	5	1	0	15
NNW	0	3	8	13	0	0	24
N	1	10	12	4	0	0	27
TOTAL	51	257	245	96	35	0	684

PERIODS OF CALM (NO. OF HOURS) – 1

MISSING DATA (NO. OF HOURS) – 35

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) SEPT 1974

TEMP. LAPSE RATE STABILITY CLASS A
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	0	0	0	0	0	0
NE	0	0	0	0	0	0	0
ENE	0	0	0	2	0	0	2
E	0	0	2	0	0	0	2
ESE	0	0	0	0	0	0	0
SE	0	0	0	0	0	0	0
SSE	0	0	0	0	0	0	0
S	0	0	2	0	0	0	2
SSW	0	13	0	0	0	0	13
SW	0	0	2	3	0	0	5
WSW	0	0	0	0	0	0	0
W	0	0	0	1	1	0	2
WNW	0	0	0	0	0	0	0
NW	0	0	0	0	0	0	0
NNW	0	0	0	0	0	0	0
N	0	0	0	0	0	0	0
TOTAL	0	13	6	6	1	0	26

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) SEPT 1974

TEMP. LAPSE RATE STABILITY CLASS B
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	0	0	0	0	0	0
NE	0	2	2	1	0	0	5
ENE	0	1	2	0	0	0	3
E	0	0	2	0	0	0	2
ESE	0	0	0	0	0	0	0
SE	0	0	0	0	0	0	0
SSE	0	0	0	0	0	0	0
S	1	0	2	0	0	0	3
SSW	0	2	3	0	0	0	5
SW	0	1	5	4	4	0	14
WSW	0	0	1	1	1	0	3
W	0	0	0	2	1	0	3
WNW	0	1	0	0	1	0	2
NW	0	0	0	2	0	0	2
NNW	0	0	0	3	0	0	3
N	0	0	0	0	0	0	0
TOTAL	1	7	17	13	7	0	45

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) SEPT 1974

TEMP. LAPSE RATE STABILITY CLASS C
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	2	0	0	0	0	2
NE	0	1	6	0	0	0	7
ENE	0	0	5	0	0	0	5
E	0	0	1	0	0	0	1
ESE	0	1	1	0	0	0	2
SE	0	0	0	0	0	0	0
SSE	0	0	0	0	0	0	0
S	1	1	1	0	0	0	3
SSW	0	0	4	1	0	0	5
SW	0	0	5	3	1	0	9
WSW	0	0	0	2	1	0	3
W	0	1	1	1	1	0	4
WNW	0	1	2	4	1	0	8
NW	0	0	1	0	0	0	1
NNW	0	1	1	1	0	0	3
N	0	0	2	0	0	0	2
TOTAL	1	8	30	12	4	0	55

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) SEPT 1974

TEMP. LAPSE RATE STABILITY CLASS D
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	2	20	26	14	10	0	72
NE	1	7	13	6	3	0	30
ENE	3	7	7	0	0	0	17
E	1	8	1	0	0	0	10
ESE	0	4	0	0	0	0	4
SE	0	0	2	0	0	0	2
SSE	1	3	5	0	0	0	9
S	1	8	7	0	0	0	16
SSW	0	16	27	2	0	0	45
SW	1	0	14	21	6	0	42
WSW	0	2	5	6	0	0	13
W	0	3	9	2	1	0	15
WNW	0	0	5	8	8	0	21
NW	0	0	5	3	1	0	9
NNW	0	1	7	9	0	0	17
N	1	10	10	4	0	0	25
TOTAL	11	103	150	60	23	0	347

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) SEPT 1974

TEMP. LAPSE RATE STABILITY CLASS E
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	0	0	0	0	0	0
NE	1	1	1	0	0	0	3
ENE	0	0	5	0	0	0	5
E	1	1	4	0	0	0	6
ESE	0	0	0	0	0	0	0
SE	0	7	0	0	0	0	7
SSE	0	11	1	0	0	0	12
S	3	17	3	0	0	0	23
SSW	1	17	10	0	0	0	28
SW	3	13	9	2	0	0	27
WSW	0	3	2	0	0	0	5
W	0	1	4	3	0	0	8
WNW	0	1	0	0	0	0	1
NW	0	0	2	0	0	0	2
NNW	0	1	0	0	0	0	1
N	0	0	0	0	0	0	0
TOTAL	9	73	41	5	0	0	128

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) SEPT 1974

TEMP. LAPSE RATE STABILITY CLASS F
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	0	0	0	0	0	0
NE	0	0	0	0	0	0	0
ENE	0	0	0	0	0	0	0
E	0	0	0	0	0	0	0
ESE	0	0	0	0	0	0	0
SE	0	2	0	0	0	0	2
SSE	1	5	0	0	0	0	6
S	4	13	0	0	0	0	17
SSW	4	7	0	0	0	0	11
SW	8	5	0	0	0	0	13
WSW	2	4	0	0	0	0	6
W	2	1	0	0	0	0	3
WNW	1	0	0	0	0	0	1
NW	0	1	0	0	0	0	1
NNW	0	0	0	0	0	0	0
N	0	0	0	0	0	0	0
TOTAL	22	38	0	0	0	0	60

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) SEPT 1974

TEMP. LAPSE RATE STABILITY CLASS G
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	0	0	0	0	0	0
NE	0	0	0	0	0	0	0
ENE	0	0	0	0	0	0	0
E	0	0	0	0	0	0	0
ESE	0	0	0	0	0	0	0
SE	0	0	0	0	0	0	0
SSE	0	2	0	0	0	0	2
S	2	7	0	0	0	0	9
SSW	2	3	1	0	0	0	6
SW	1	3	0	0	0	0	4
WSW	2	0	0	0	0	0	2
W	0	0	0	0	0	0	0
WNW	0	0	0	0	0	0	0
NW	0	0	0	0	0	0	0
NNW	0	0	0	0	0	0	0
N	0	0	0	0	0	0	0
TOTAL	7	15	1	0	0	0	23

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

24-HOUR SUMMARY OF WIND SPEED DISTRIBUTION

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) OCT 1974

TOTAL NUMBER OF READINGS	7.32000E+02
TOTAL NUMBER OF READINGS WITHOUT CALMS	7.29000E+02

****

WIND SPEED DISTRIBUTION, NO OF OBS.

LT .5	.50 – 3.5	3.51 – 7.5	7.51 – 12.5	12.51 – 18.5	18.51 – 24.0	GT 24.0
3	51	259	278	130	11	0

****

SUMMED OVER ALL DIRECTIONS
WIND SPEED DISTRIBUTION VERSUS TEMP. LAPSE RATE STABILITY CLASS (NO OF OBS.)

	A	B	C	D	E	F	G
1	0	0	0	0	3	0	0
2	0	0	0	7	25	12	7
3	3	3	11	92	59	47	44
4	5	18	26	135	86	4	4
5	1	23	10	86	10	0	0
6	0	0	3	8	0	0	0
7	0	0	0	1	0	0	0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) OCT 1974

****

SUMMED OVER ALL TEMP. LAPSE RATE STABILITIES
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	2	14	18	11	0	0	45
NE	4	12	7	5	0	0	28
ENE	2	4	1	0	0	0	7
E	3	10	3	0	0	0	16
ESE	3	14	2	0	0	0	19
SE	3	14	2	0	0	0	21
SSE	7	8	1	0	0	0	16
S	6	33	23	0	0	0	62
SSW	6	54	73	0	0	0	156
SW	2	36	41	28	2	0	109
WSW	3	19	25	9	4	0	60
W	3	7	12	11	0	0	33
WNW	2	2	4	6	0	0	14
NW	1	6	9	15	1	0	32
NNW	0	8	30	15	3	0	56
N	2	18	27	7	1	0	55
TOTAL	51	259	278	130	11	0	729

PERIODS OF CALM (NO. OF HOURS) – 3

MISSING DATA (NO. OF HOURS) – 12

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) OCT 1974

TEMP. LAPSE RATE STABILITY CLASS A
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	0	0	0	0	0	0
NE	0	0	1	0	0	0	1
ENE	0	0	0	0	0	0	0
E	0	0	0	0	0	0	0
ESE	0	0	0	0	0	0	0
SE	0	0	0	0	0	0	0
SSE	0	0	0	0	0	0	0
S	0	0	0	0	0	0	0
SSW	0	2	3	0	0	0	5
SW	0	0	1	0	0	0	1
WSW	0	1	0	0	0	0	1
W	0	0	0	0	0	0	0
WNW	0	0	0	1	0	0	1
NW	0	0	0	0	0	0	0
NNW	0	0	0	0	0	0	0
N	0	0	0	0	0	0	0
TOTAL	0	3	5	1	0	0	9

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) OCT 1974

TEMP. LAPSE RATE STABILITY CLASS B
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND DIRECTION	WIND SPEED (MPH) AT 10 METER LEVEL 1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	0	0	0	0	0	0
NE	0	0	0	1	0	0	1
ENE	0	1	0	0	0	0	1
E	0	1	1	0	0	0	2
ESE	0	0	0	0	0	0	0
SE	0	0	0	0	0	0	0
SSE	0	0	0	0	0	0	0
S	0	0	1	0	0	0	1
SSW	0	0	6	5	0	0	11
SW	0	0	5	8	0	0	13
WSW	0	1	3	1	0	0	5
W	0	0	1	5	0	0	6
WNW	0	0	0	0	0	0	0
NW	0	0	1	3	0	0	4
NNW	0	0	0	0	0	0	0
N	0	0	0	0	0	0	0
TOTAL	0	3	18	23	0	0	44

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) OCT 1974

TEMP. LAPSE RATE STABILITY CLASS C
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	0	0	0	0	0	0
NE	0	2	0	0	0	0	2
ENE	0	0	0	0	0	0	0
E	0	2	1	0	0	0	3
ESE	0	2	0	0	0	0	2
SE	0	1	0	0	0	0	1
SSE	0	0	0	0	0	0	0
S	0	0	0	0	0	0	0
SSW	0	1	6	3	0	0	10
SW	0	0	6	2	1	0	9
WSW	0	1	0	2	2	0	5
W	0	0	2	0	0	0	2
WNW	0	0	1	0	0	0	1
NW	0	1	3	1	0	0	5
NNW	0	0	7	2	0	0	9
N	0	1	0	0	0	0	1
TOTAL	0	11	26	10	3	0	50

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) OCT 1974

TEMP. LAPSE RATE STABILITY CLASS D
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	13	18	11	0	0	42
NE	0	7	5	4	0	0	16
ENE	1	2	1	0	0	0	4
E	1	4	1	0	0	0	6
ESE	2	7	2	0	0	0	11
SE	0	3	1	0	0	0	4
SSE	1	0	1	0	0	0	2
S	0	4	5	0	0	0	9
SSW	0	9	19	12	0	0	40
SW	0	5	11	12	1	0	29
WSW	0	6	11	5	2	24	24
W	0	3	9	6	0	0	18
WNW	0	1	2	5	0	0	8
NW	0	4	4	11	1	0	20
NNW	0	8	20	13	3	0	44
N	2	16	25	7	1	0	51
TOTAL	7	92	135	86	8	0	328

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) OCT 1974

TEMP. LAPSE RATE STABILITY CLASS E
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	2	1	0	0	0	0	3
NE	4	3	1	0	0	0	8
ENE	1	1	0	0	0	0	2
E	1	3	0	0	0	0	4
ESE	0	1	0	0	0	0	1
SE	2	2	1	0	0	0	5
SSE	4	2	0	0	0	0	6
S	3	11	17	0	0	0	31
SSW	3	13	36	3	0	0	55
SW	0	14	14	6	0	0	34
WSW	2	5	11	1	0	0	19
W	1	1	0	0	0	0	2
WNW	1	0	0	0	0	0	1
NW	1	1	1	0	0	0	3
NNW	0	0	3	0	0	0	3
N	0	1	2	0	0	0	3
TOTAL	25	59	86	10	0	0	180

PERIODS OF CALM (NO. OF HOURS) – 3

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) OCT 1974

TEMP. LAPSE RATE STABILITY CLASS F
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	0	0	0	0	0	0
NE	0	0	0	0	0	0	0
ENE	0	0	0	0	0	0	0
E	1	0	0	0	0	0	1
ESE	1	0	0	0	0	0	1
SE	1	5	0	0	0	0	6
SSE	1	5	0	0	0	0	6
S	1	10	0	0	0	0	11
SSW	1	15	3	0	0	0	19
SW	2	6	1	0	0	0	9
WSW	1	2	0	0	0	0	3
W	2	3	0	0	0	0	5
WNW	1	1	0	0	0	0	2
NW	0	0	0	0	0	0	0
NNW	0	0	0	0	0	0	0
N	0	0	0	0	0	0	0
TOTAL	12	47	4	0	0	0	63

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) OCT 1974

TEMP. LAPSE RATE STABILITY CLASS G
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	0	0	0	0	0	0
NE	0	0	0	0	0	0	0
ENE	0	0	0	0	0	0	0
E	0	0	0	0	0	0	0
ESE	0	4	0	0	0	0	4
SE	2	3	0	0	0	0	5
SSE	1	1	0	0	0	0	2
S	2	8	0	0	0	0	10
SSW	2	14	0	0	0	0	16
SW	0	11	3	0	0	0	14
WSW	0	3	0	0	0	0	3
W	0	0	0	0	0	0	0
WNW	0	0	1	0	0	0	1
NW	0	0	0	0	0	0	0
NNW	0	0	0	0	0	0	0
N	0	0	0	0	0	0	0
TOTAL	7	44	4	0	0	0	55

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

24 Hour Summary of Wind Speed Distribution

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) NOV 1974

TOTAL NUMBER OF READINGS	6.76000E+02
TOTAL NUMBER OF READINGS WITHOUT CALMS	6.75000E+02

****

Wind Speed Distribution, No of Obs.

LT .5	.50 – 3.5	3.51 – 7.5	7.51 – 12.5	12.51 – 18.5	18.51 – 24.0	GT 24.0
1	36	210	187	167	66	9

****

SUMMED OVER ALL DIRECTIONS
WIND SPEED DISTRIBUTION VERSUS TEMP. LAPSE RATE STABILITY CLASS (NO OF OBS.)

	A	B	C	D	E	F	G
1	0	0	0	0	1	0	0
2	3	0	1	16	9	1	6
3	10	2	4	78	66	39	11
4	2	6	10	102	58	9	0
5	3	8	9	135	11	1	0
6	0	0	1	65	0	0	0
7	0	0	0	9	0	0	0

>

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) NOV 1974

****

SUMMED OVER ALL TEMP. LAPSE RATE STABILITIES
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND DIRECTION	WIND SPEED (MPH) AT 10 METER LEVEL 1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	1	2	0	3	0	0	6
NE	1	6	2	10	0	0	19
ENE	2	5	4	6	4	2	23
E	1	7	2	0	9	1	20
ESE	1	8	4	4	1	0	18
SE	4	12	5	1	0	0	22
SSE	2	14	1	2	0	0	19
S	4	39	6	1	0	0	50
SSW	6	27	37	24	1	0	95
SW	6	28	32	18	7	0	91
WSW	2	20	20	45	15	5	107
W	2	23	29	21	14	0	89
WNW	0	7	10	4	6	1	28
NW	2	6	25	5	7	0	45
NNW	2	1	8	15	1	0	27
N	0	5	2	8	1	0	16
TOTAL	36	210	187	167	66	9	675

PERIODS OF CALM (NO. OF HOURS) – 1

MISSING DATA (NO. OF HOURS) – 44

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) NOV 1974

TEMP. LAPSE RATE STABILITY CLASS A
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	1	0	0	0	0	1
NE	0	0	0	0	0	0	0
ENE	0	0	0	0	0	0	0
E	1	1	1	0	0	0	3
ESE	0	2	0	0	0	0	2
SE	0	2	0	0	0	0	2
SSE	0	2	0	0	0	0	2
S	0	0	0	0	0	0	0
SSW	0	1	0	0	0	0	1
SW	0	0	0	0	0	0	0
WSW	0	1	0	0	0	0	1
W	1	0	0	0	0	0	1
WNW	0	0	0	0	0	0	0
NW	1	0	1	1	0	0	3
NNW	0	0	0	2	0	0	2
N	0	0	0	0	0	0	0
TOTAL	3	10	2	3	0	0	18

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) NOV 1974

TEMP. LAPSE RATE STABILITY CLASS B
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	0	0	1	0	0	1
NE	0	0	1	4	0	0	5
ENE	0	1	0	0	0	0	1
E	0	0	0	0	0	0	0
ESE	0	0	0	0	0	0	0
SE	0	0	0	0	0	0	0
SSE	0	0	0	0	0	0	0
S	0	0	0	1	0	0	1
SSW	0	0	0	0	0	0	0
SW	0	0	0	0	0	0	0
WSW	0	0	0	1	0	0	1
W	0	0	2	1	0	0	3
WNW	0	1	1	0	0	0	2
NW	0	0	2	0	0	0	2
NNW	0	0	0	0	0	0	0
N	0	0	0	0	0	0	0
TOTAL	0	2	6	8	0	0	16

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) NOV 1974

TEMP. LAPSE RATE STABILITY CLASS C
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	0	0	1	0	0	1
NE	0	0	0	1	0	0	1
ENE	0	0	0	0	0	0	0
E	0	0	0	0	0	0	0
ESE	0	0	0	0	0	0	0
SE	1	0	0	0	0	0	1
SSE	0	0	0	0	0	0	0
S	0	0	0	0	0	0	0
SSW	0	1	3	3	0	0	7
SW	0	2	1	1	0	0	4
WSW	0	1	2	1	1	0	5
W	0	0	0	1	0	0	1
WNW	0	0	0	0	0	0	0
NW	0	0	2	0	0	0	2
NNW	0	0	1	1	0	0	2
N	0	0	1	0	0	0	1
TOTAL	1	4	10	9	1	0	25

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) NOV 1974

TEMP. LAPSE RATE STABILITY CLASS D
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	1	1	0	1	0	0	3
NE	1	5	1	5	0	0	12
ENE	1	2	1	6	4	2	16
E	0	4	0	0	9	1	14
ESE	0	2	2	4	1	0	9
SE	1	2	2	1	0	0	6
SSE	2	4	0	2	0	0	8
S	2	9	2	0	0	0	13
SSW	2	5	12	16	1	0	36
SW	2	5	7	17	7	0	38
WSW	1	8	17	38	14	5	83
W	1	16	21	18	14	0	70
WNW	0	4	9	4	6	1	24
NW	1	6	20	4	7	0	38
NNW	1	1	7	11	1	0	21
N	0	4	1	8	1	0	14
TOTAL	16	78	102	135	65	9	405

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) NOV 1974

TEMP. LAPSE RATE STABILITY CLASS E
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	0	0	0	0	0	0
NE	0	1	0	0	0	0	1
ENE	1	2	3	0	0	0	6
E	0	2	1	0	0	0	3
ESE	1	3	2	0	0	0	6
SE	2	5	3	0	0	0	10
SSE	0	6	1	0	0	0	7
S	1	15	4	0	0	0	20
SSW	2	13	17	4	0	0	36
SW	0	7	20	0	0	0	27
WSW	1	3	1	5	0	0	10
W	0	7	6	1	0	0	14
WNW	0	1	0	0	0	0	1
NW	0	0	0	0	0	0	0
NNW	1	0	0	1	0	0	2
N	0	1	0	0	0	0	1
TOTAL	9	66	58	11	0	0	144

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) NOV 1974

TEMP. LAPSE RATE STABILITY CLASS F
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	0	0	0	0	0	0
NE	0	0	0	0	0	0	0
ENE	0	0	0	0	0	0	0
E	0	0	0	0	0	0	0
ESE	0	0	0	0	0	0	0
SE	0	3	0	0	0	0	3
SSE	0	1	0	0	0	0	1
S	0	12	0	0	0	0	12
SSW	1	7	5	1	0	0	14
SW	0	9	4	0	0	0	13
WSW	0	5	0	0	0	0	5
W	0	0	0	0	0	0	0
WNW	0	1	0	0	0	0	1
NW	0	0	0	0	0	0	0
NNW	0	0	0	0	0	0	0
N	0	0	0	0	0	0	0
TOTAL	1	39	9	1	0	0	50

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) NOV 1974

TEMP. LAPSE RATE STABILITY CLASS G
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	0	0	0	0	0	0
NE	0	0	0	0	0	0	0
ENE	0	0	0	0	0	0	0
E	0	0	0	0	0	0	0
ESE	0	0	0	0	0	0	0
SE	0	0	0	0	0	0	0
SSE	0	1	0	0	0	0	1
S	1	3	0	0	0	0	4
SSW	1	0	0	0	0	0	1
SW	4	5	0	0	0	0	9
WSW	0	2	0	0	0	0	2
W	0	0	0	0	0	0	0
WNW	0	0	0	0	0	0	0
NW	0	0	0	0	0	0	0
NNW	0	0	0	0	0	0	0
N	0	0	0	0	0	0	0
TOTAL	6	11	0	0	0	0	17

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

24 – Hour Summary of Wind Speed Distribution

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) DEC 1974

TOTAL NUMBER OF READINGS	6.08000E+02
TOTAL NUMBER OF READINGS WITHOUT CALMS	6.08000E+02

****

WIND SPEED DISTRIBUTION, NO OF OBS.

LT .5	.50 – 3.5	3.51 – 7.5	7.51 – 12.5	12.51 – 18.5	18.51 – 24.0	GT 24.0
0	80	198	193	113	18	6

****

SUMMED OVER ALL DIRECTIONS
WIND SPEED DISTRIBUTION VERSUS TEMP. LAPSE RATE STABILITY CLASS (NO OF OBS.)

	A	B	C	D	E	F	G
1	0	0	0	0	0	0	0
2	0	0	0	16	47	7	10
3	0	0	0	52	95	47	4
4	0	1	1	92	89	9	1
5	0	0	0	84	26	3	0
6	0	0	0	15	3	0	0
7	0	0	0	3	3	0	0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) DEC 1974

****

SUMMED OVER ALL TEMP. LAPSE RATE STABILITIES
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	3	2	5	1	0	0	11
NE	0	2	7	0	0	0	9
ENE	1	9	7	0	0	0	17
E	6	11	6	1	6	4	34
ESE	4	10	7	4	0	0	25
SE	8	21	8	5	0	0	42
SSE	8	10	11	5	0	0	34
S	11	22	8	1	0	0	42
SSW	13	47	43	4	2	0	109
SW	6	34	42	26	0	0	108
WSW	5	8	24	40	5	2	84
W	2	8	12	19	5	0	46
WNW	5	3	3	4	0	0	15
NW	3	6	6	2	0	1	17
NNW	4	5	1	0	0	0	10
N	1	0	3	1	0	0	5
TOTAL	80	198	193	113	18	6	608

PERIODS OF CALM (NO. OF HOURS) – 0

MISSING DATA (NO. OF HOURS) – 136

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) DEC 1974

TEMP. LAPSE RATE STABILITY CLASS A
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	0	0	0	0	0	0
NE	0	0	0	0	0	0	0
ENE	0	0	0	0	0	0	0
E	0	0	0	0	0	0	0
ESE	0	0	0	0	0	0	0
SE	0	0	0	0	0	0	0
SSE	0	0	0	0	0	0	0
S	0	0	0	0	0	0	0
SSW	0	0	0	0	0	0	0
SW	0	0	0	0	0	0	0
WSW	0	0	0	0	0	0	0
W	0	0	0	0	0	0	0
WNW	0	0	0	0	0	0	0
NW	0	0	0	0	0	0	0
NNW	0	0	0	0	0	0	0
N	0	0	0	0	0	0	0
TOTAL	0	0	0	0	0	0	0

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) DEC 1974

TEMP. LAPSE RATE STABILITY CLASS B
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	0	0	0	0	0	0
NE	0	0	0	0	0	0	0
ENE	0	0	0	0	0	0	0
E	0	0	0	0	0	0	0
ESE	0	0	0	0	0	0	0
SE	0	0	0	0	0	0	0
SSE	0	0	0	0	0	0	0
S	0	0	0	0	0	0	0
SSW	0	0	0	0	0	0	0
SW	0	0	0	0	0	0	0
WSW	0	0	0	0	0	0	0
W	0	0	0	0	0	0	0
WNW	0	0	0	0	0	0	0
NW	0	0	0	0	0	0	0
NNW	0	0	0	0	0	0	0
N	0	0	1	0	0	0	1
TOTAL	0	0	1	0	0	0	1

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) DEC 1974

TEMP. LAPSE RATE STABILITY CLASS C
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	0	0	0	0	0	0
NE	0	0	0	0	0	0	0
ENE	0	0	0	0	0	0	0
E	0	0	0	0	0	0	0
ESE	0	0	0	0	0	0	0
SE	0	0	0	0	0	0	0
SSE	0	0	0	0	0	0	0
S	0	0	0	0	0	0	0
SSW	0	0	0	0	0	0	0
SW	0	0	0	0	0	0	0
WSW	0	0	0	0	0	0	0
W	0	0	0	0	0	0	0
WNW	0	0	0	0	0	0	0
NW	0	0	0	0	0	0	0
NNW	0	0	0	0	0	0	0
N	0	0	1	0	0	0	1
TOTAL	0	0	1	0	0	0	1

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) DEC 1974

TEMP. LAPSE RATE STABILITY CLASS D
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	1	0	0	0	0	0	1
NE	0	1	3	0	0	0	4
ENE	0	6	3	0	0	0	9
E	2	5	1	1	4	1	14
ESE	0	0	1	0	0	0	1
SE	0	4	2	1	0	0	7
SSE	0	0	5	0	0	0	5
S	0	5	3	0	0	0	8
SSW	3	6	17	0	1	0	27
SW	0	10	24	21	0	0	55
WSW	2	4	20	36	5	2	69
W	2	4	8	18	5	0	37
WNW	3	1	1	4	0	0	9
NW	2	2	3	2	0	0	9
NNW	1	4	0	0	0	0	5
N	0	0	1	1	0	0	2
TOTAL	16	52	92	84	15	3	262

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) DEC 1974

TEMP. LAPSE RATE STABILITY CLASS E
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 – METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	2	2	5	1	0	0	10
NE	0	1	4	0	0	0	5
ENE	1	3	4	0	0	0	8
E	3	6	3	0	2	3	17
ESE	2	6	6	4	0	0	18
SE	6	9	6	4	0	0	25
SSE	4	5	6	5	0	0	20
S	6	14	5	1	0	0	26
SSW	7	21	21	3	1	0	53
SW	6	15	16	3	0	0	40
WSW	3	4	4	4	0	0	15
W	0	4	4	1	0	0	9
WNW	2	2	2	0	0	0	6
NW	1	3	3	0	0	0	7
NNW	3	0	0	0	0	0	3
N	1	0	0	0	0	0	1
TOTAL	47	95	89	26	3	3	263

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) DEC 1974

TEMP. LAPSE RATE STABILITY CLASS F
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	0	0	0	0	0	0
NE	0	0	0	0	0	0	0
ENE	0	0	0	0	0	0	0
E	0	0	2	0	0	0	2
ESE	2	4	0	0	0	0	6
SE	2	6	0	0	0	0	8
SSE	1	5	0	0	0	0	6
S	0	3	0	0	0	0	3
SSW	2	18	4	1	0	0	25
SW	0	9	2	0	0	0	11
WSW	0	0	0	0	0	0	0
W	0	0	0	0	0	0	0
WNW	0	0	0	0	0	0	0
NW	0	1	0	0	0	0	1
NNW	0	1	1	0	0	0	2
N	0	0	0	0	0	0	0
TOTAL	7	47	9	1	0	0	64

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) DEC 1974

TEMP. LAPSE RATE STABILITY CLASS G
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	0	0	0	0	0	0
NE	0	0	0	0	0	0	0
ENE	0	0	0	0	0	0	0
E	1	0	0	0	0	0	1
ESE	0	0	0	0	0	0	0
SE	0	2	0	0	0	0	2
SSE	3	0	0	0	0	0	3
S	5	0	0	0	0	0	5
SSW	1	2	1	0	0	0	4
SW	0	0	0	0	0	0	0
WSW	0	0	0	0	0	0	0
W	0	0	0	0	0	0	0
WNW	0	0	0	0	0	0	0
NW	0	0	0	0	0	0	0
NNW	0	0	0	0	0	0	0
N	0	0	0	0	0	0	0
TOTAL	10	4	1	0	0	0	15

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

24 – HOUR SUMMARY OF WIND SPEED DISTRIBUTION

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) JAN 1975

TOTAL NUMBER OF READINGS 7.31000E+02

TOTAL NUMBER OF READINGS WITHOUT CALMS 7.31000E+02

****

WIND SPEED DISTRIBUTION, NO OF OBS.

LT .5	.50 – 3.5	3.51 – 7.5	7.51 – 12.5	12.51 – 18.5	18.51 – 24.0	GT 24.0
0	31	126	262	185	92	35

****

SUMMED OVER ALL DIRECTIONS
WIND SPEED DISTRIBUTION VERSUS TEMP. LAPSE RATE STABILITY CLASS (NO OF OBS.)

	A	B	C	D	E	F	G
1	0	0	0	0	0	0	0
2	0	0	0	17	13	1	0
3	0	0	0	54	55	17	0
4	0	0	0	144	169	8	1
5	0	0	0	123	62	0	0
6	0	0	0	57	34	1	0
7	0	0	0	29	6	0	0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) JAN 1975

****

SUMMED OVER ALL TEMP. LAPSE RATE STABILITIES
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	2	6	10	5	0	23
NE	3	4	8	5	0	0	20
ENE	1	2	10	5	0	0	18
E	2	3	8	1	0	0	14
ESE	1	9	13	0	0	0	23
SE	2	8	10	0	0	1	21
SSE	3	8	8	5	10	0	34
S	7	15	32	7	3	1	65
SSW	1	21	38	18	13	0	91
SW	2	18	43	32	9	10	114
WSW	2	13	44	51	32	12	154
W	3	6	23	35	18	10	95
WNW	0	3	5	14	2	1	25
NW	0	4	9	2	0	0	15
NNW	1	3	1	0	0	0	5
N	3	7	4	0	0	0	14
TOTAL	31	126	262	185	92	35	731

PERIODS OF CALM (NO. OF HOURS) – 0

MISSING DATA (NO. OF HOURS) – 13

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) JAN 1975

TEMP. LAPSE RATE STABILITY CLASS A
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND DIRECTION	WIND SPEED (MPH) AT 10 METER LEVEL 1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	0	0	0	0	0	0
NE	0	0	0	0	0	0	0
ENE	0	0	0	0	0	0	0
E	0	0	0	0	0	0	0
ESE	0	0	0	0	0	0	0
SE	0	0	0	0	0	0	0
SSE	0	0	0	0	0	0	0
S	0	0	0	0	0	0	0
SSW	0	0	0	0	0	0	0
SW	0	0	0	0	0	0	0
WSW	0	0	0	0	0	0	0
W	0	0	0	0	0	0	0
WNW	0	0	0	0	0	0	0
NW	0	0	0	0	0	0	0
NNW	0	0	0	0	0	0	0
N	0	0	0	0	0	0	0
TOTAL	0	0	0	0	0	0	0

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) JAN 1975

TEMP. LAPSE RATE STABILITY CLASS B
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	0	0	0	0	0	0
NE	0	0	0	0	0	0	0
ENE	0	0	0	0	0	0	0
E	0	0	0	0	0	0	0
ESE	0	0	0	0	0	0	0
SE	0	0	0	0	0	0	0
SSE	0	0	0	0	0	0	0
S	0	0	0	0	0	0	0
SSW	0	0	0	0	0	0	0
SW	0	0	0	0	0	0	0
WSW	0	0	0	0	0	0	0
W	0	0	0	0	0	0	0
WNW	0	0	0	0	0	0	0
NW	0	0	0	0	0	0	0
NNW	0	0	0	0	0	0	0
N	0	0	0	0	0	0	0
TOTAL	0	0	0	0	0	0	0

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) JAN 1975

TEMP. LAPSE RATE STABILITY CLASS C
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	0	0	0	0	0	0
NE	0	0	0	0	0	0	0
ENE	0	0	0	0	0	0	0
E	0	0	0	0	0	0	0
ESE	0	0	0	0	0	0	0
SE	0	0	0	0	0	0	0
SSE	0	0	0	0	0	0	0
S	0	0	0	0	0	0	0
SSW	0	0	0	0	0	0	0
SW	0	0	0	0	0	0	0
WSW	0	0	0	0	0	0	0
W	0	0	0	0	0	0	0
WNW	0	0	0	0	0	0	0
NW	0	0	0	0	0	0	0
NNW	0	0	0	0	0	0	0
N	0	0	0	0	0	0	0
TOTAL	0	0	0	0	0	0	0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) JAN 1975

TEMP. LAPSE RATE STABILITY CLASS D
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND DIRECTION	WIND SPEED (MPH) AT 10 METER LEVEL 1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	2	6	10	5	0	23
NE	2	2	8	5	0	0	17
ENE	0	2	10	5	0	0	17
E	0	0	4	1	0	0	5
ESE	0	1	0	0	0	0	1
SE	1	0	2	0	0	0	3
SSE	1	5	1	2	0	0	9
S	3	5	10	4	0	0	22
SSW	1	8	23	5	10	0	47
SW	1	10	18	13	2	8	52
WSW	2	6	32	39	22	11	112
W	3	3	15	26	16	10	73
WNW	0	0	2	11	2	0	15
NW	0	3	8	2	0	0	13
NNW	1	2	1	0	0	0	4
N	2	5	4	0	0	0	11
TOTAL	17	54	144	123	57	29	424

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) JAN 1975

TEMP. LAPSE RATE STABILITY CLASS E
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND DIRECTION	WIND SPEED (MPH) AT 10 METER LEVEL 1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	0	0	0	0	0	0
NE	1	2	0	0	0	0	3
ENE	1	0	0	0	0	0	1
E	2	2	2	0	0	0	6
ESE	1	5	10	0	0	0	16
SE	1	4	6	0	0	1	12
SSE	2	2	5	3	9	0	21
S	3	7	22	3	3	1	39
SSW	0	9	15	13	3	0	40
SW	1	8	25	19	7	2	62
WSW	0	7	12	12	10	1	42
W	0	3	8	9	2	0	22
WNW	0	3	3	3	0	1	10
NW	0	0	1	0	0	0	1
NNW	0	1	0	0	0	0	1
N	1	2	0	0	0	0	3
TOTAL	13	55	109	62	34	6	279

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) JAN 1975

TEMP. LAPSE RATE STABILITY CLASS F
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL					
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	0	0	0	0	0	0
NE	0	0	0	0	0	0	0
ENE	0	0	0	0	0	0	0
E	0	1	2	0	0	0	3
ESE	0	3	2	0	0	0	5
SE	0	4	2	0	0	0	6
SSE	0	1	2	0	1	0	4
S	1	3	0	0	0	0	4
SSW	0	4	0	0	0	0	4
SW	0	0	0	0	0	0	0
WSW	0	0	0	0	0	0	0
W	0	0	0	0	0	0	0
WNW	0	0	0	0	0	0	0
NW	0	1	0	0	0	0	1
NNW	0	0	0	0	0	0	0
N	0	0	0	0	0	0	0
TOTAL	1	17	8	0	1	0	27

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) JAN 1975

TEMP. LAPSE RATE STABILITY CLASS G
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	0	0	0	0	0	0
NE	0	0	0	0	0	0	0
ENE	0	0	0	0	0	0	0
E	0	0	0	0	0	0	0
ESE	0	0	1	0	0	0	1
SE	0	0	0	0	0	0	0
SSE	0	0	0	0	0	0	0
S	0	0	0	0	0	0	0
SSW	0	0	0	0	0	0	0
SW	0	0	0	0	0	0	0
WSW	0	0	0	0	0	0	0
W	0	0	0	0	0	0	0
WNW	0	0	0	0	0	0	0
NW	0	0	0	0	0	0	0
NNW	0	0	0	0	0	0	0
N	0	0	0	0	0	0	0
TOTAL	0	0	1	0	0	0	1

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

24 Hour Summary of Wind Speed Distribution

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) FEB 1975

TOTAL NUMBER OF READINGS 5.94000E+02

TOTAL NUMBER OF READINGS WITHOUT CALMS 5.94000E+02

****

WIND SPEED DISTRIBUTION, NO OF OBS.

LT .5	.50 – 3.5	3.51 – 7.5	7.51 – 12.5	12.51 – 18.5	18.51 – 24.0	GT 24.0
0	19	105	211	190	39	30

****

SUMMED OVER ALL DIRECTIONS
WIND SPEED DISTRIBUTION VERSUS TEMP. LAPSE RATE STABILITY CLASS (NO OF OBS.)

	A	B	C	D	E	F	G
1	0	0	0	0	0	0	0
2	0	0	0	6	8	3	2
3	0	0	0	43	48	10	4
4	0	0	0	126	71	11	3
5	0	1	1	152	36	0	0
6	0	0	0	29	10	0	0
7	0	0	0	20	10	0	0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) FEB 1975

****

SUMMED OVER ALL TEMP. LAPSE RATE STABILITIES
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	2	6	3	0	0	11
NE	2	3	22	20	0	0	47
ENE	1	5	20	1	0	0	27
E	4	10	15	10	0	0	39
ESE	1	8	15	12	0	0	36
SE	0	6	11	1	0	0	18
SSE	1	8	6	0	0	0	15
S	0	9	4	0	0	0	13
SSW	3	10	19	9	0	0	41
SW	3	13	36	25	10	4	91
WSW	2	11	19	33	8	22	95
W	0	4	19	44	16	4	87
WNW	1	5	9	17	4	0	36
NW	0	3	7	12	1	0	23
NNW	0	5	2	2	0	0	9
N	1	3	1	1	0	0	6
TOTAL	19	105	211	190	39	30	594

PERIODS OF CALM (NO. OF HOURS) – 0

MISSING DATA (NO. OF HOURS) – 78

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) FEB 1975

TEMP. LAPSE RATE STABILITY CLASS A
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	0	0	0	0	0	0
NE	0	0	0	0	0	0	0
ENE	0	0	0	0	0	0	0
E	0	0	0	0	0	0	0
ESE	0	0	0	0	0	0	0
SE	0	0	0	0	0	0	0
SSE	0	0	0	0	0	0	0
S	0	0	0	0	0	0	0
SSW	0	0	0	0	0	0	0
SW	0	0	0	0	0	0	0
WSW	0	0	0	0	0	0	0
W	0	0	0	0	0	0	0
WNW	0	0	0	0	0	0	0
NW	0	0	0	0	0	0	0
NNW	0	0	0	0	0	0	0
N	0	0	0	0	0	0	0
TOTAL	0	0	0	0	0	0	0

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) FEB 1975

TEMP. LAPSE RATE STABILITY CLASS B
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	0	0	0	0	0	0
NE	0	0	0	0	0	0	0
ENE	0	0	0	0	0	0	0
E	0	0	0	1	0	0	1
ESE	0	0	0	0	0	0	0
SE	0	0	0	0	0	0	0
SSE	0	0	0	0	0	0	0
S	0	0	0	0	0	0	0
SSW	0	0	0	0	0	0	0
SW	0	0	0	0	0	0	0
WSW	0	0	0	0	0	0	0
W	0	0	0	0	0	0	0
WNW	0	0	0	0	0	0	0
NW	0	0	0	0	0	0	0
NNW	0	0	0	0	0	0	0
N	0	0	0	0	0	0	0
TOTAL	0	0	0	1	0	0	1

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) FEB 1975

TEMP. LAPSE RATE STABILITY CLASS C
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	0	0	0	0	0	0
NE	0	0	0	0	0	0	0
ENE	0	0	0	0	0	0	0
E	0	0	0	1	0	0	1
ESE	0	0	0	0	0	0	0
SE	0	0	0	0	0	0	0
SSE	0	0	0	0	0	0	0
S	0	0	0	0	0	0	0
SSW	0	0	0	0	0	0	0
SW	0	0	0	0	0	0	0
WSW	0	0	0	0	0	0	0
W	0	0	0	0	0	0	0
WNW	0	0	0	0	0	0	0
NW	0	0	0	0	0	0	0
NNW	0	0	0	0	0	0	0
N	0	0	0	0	0	0	0
TOTAL	0	0	0	1	0	0	1

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) FEB 1975

TEMP. LAPSE RATE STABILITY CLASS D
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	2	6	3	0	0	11
NE	0	2	18	19	0	0	39
ENE	1	2	14	0	0	0	17
E	3	8	12	6	0	0	29
ESE	0	7	12	11	0	0	30
SE	0	4	6	1	0	0	11
SSE	0	4	0	0	0	0	4
S	0	1	0	0	0	0	1
SSW	0	2	8	6	0	0	16
SW	1	4	19	23	7	3	57
WSW	1	1	4	21	4	13	44
W	0	1	12	35	13	4	65
WNW	0	2	9	13	4	0	28
NW	0	1	5	11	1	0	18
NNW	0	1	1	2	0	0	4
N	0	1	0	1	0	0	2
TOTAL	6	43	126	152	29	20	376

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) FEB 1975

TEMP. LAPSE RATE STABILITY CLASS E
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	0	0	0	0	0	0
NE	1	1	4	1	0	0	7
ENE	0	3	5	1	0	0	9
E	0	1	0	2	0	0	3
ESE	1	1	1	1	0	0	4
SE	0	1	2	0	0	0	3
SSE	0	3	4	0	0	0	7
S	0	5	4	0	0	0	9
SSW	2	5	11	3	0	0	21
SW	2	9	15	2	3	1	32
WSW	1	7	15	12	4	9	48
W	0	3	7	9	3	0	22
WNW	1	3	0	4	0	0	8
NW	0	2	1	1	0	0	4
NNW	0	3	1	0	0	0	4
N	0	1	1	0	0	0	2
TOTAL	8	48	71	36	10	10	183

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) FEB 1975

TEMP. LAPSE RATE STABILITY CLASS F
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	0	0	0	0	0	0
NE	0	0	0	0	0	0	0
ENE	0	0	0	0	0	0	0
E	1	0	2	0	0	0	3
ESE	0	0	1	0	0	0	1
SE	0	0	3	0	0	0	3
SSE	0	1	2	0	0	0	3
S	0	3	0	0	0	0	3
SSW	1	3	0	0	0	0	4
SW	0	0	2	0	0	0	2
WSW	0	2	0	0	0	0	2
W	0	0	0	0	0	0	0
WNW	0	0	0	0	0	0	0
NW	0	0	1	0	0	0	1
NNW	0	0	0	0	0	0	0
N	1	1	0	0	0	0	2
TOTAL	3	10	11	0	0	0	24

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) FEB 1975

TEMP. LAPSE RATE STABILITY CLASS G
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	0	0	0	0	0	0
NE	1	0	0	0	0	0	1
ENE	0	0	1	0	0	0	1
E	0	1	1	0	0	0	2
ESE	0	0	1	0	0	0	1
SE	0	1	0	0	0	0	1
SSE	1	0	0	0	0	0	1
S	0	0	0	0	0	0	0
SSW	0	0	0	0	0	0	0
SW	0	0	0	0	0	0	0
WSW	0	1	0	0	0	0	1
W	0	0	0	0	0	0	0
WNW	0	0	0	0	0	0	0
NW	0	0	0	0	0	0	0
NNW	0	1	0	0	0	0	1
N	0	0	0	0	0	0	0
TOTAL	2	4	3	0	0	0	9

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

24 Hour Summary of Wind Speed Distribution

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) MAR 1975

TOTAL NUMBER OF READINGS	6.30000E+02
TOTAL NUMBER OF READINGS WITHOUT CALMS	6.30000E+02

****

WIND SPEED DISTRIBUTION, NO OF OBS.

LT .5	.50 – 3.5	3.51 – 7.5	7.51 – 12.5	12.51 – 18.5	18.51 – 24.0	GT 24.0
0	18	73	213	221	85	20

****

SUMMED OVER ALL DIRECTIONS
WIND SPEED DISTRIBUTION VERSUS TEMP. LAPSE RATE STABILITY CLASS (NO OF OBS.)

	A	B	C	D	E	F	G
1	0	0	0	0	0	0	0
2	0	0	0	4	8	5	1
3	0	0	0	21	40	6	6
4	0	0	0	116	76	13	8
5	0	0	0	175	43	3	0
6	0	0	3	72	10	0	0
7	0	0	0	20	0	0	0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) MAR 1975

****

SUMMED OVER ALL TEMP. LAPSE RATE STABILITIES
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	1	9	1	1	0	12
NE	0	6	20	0	0	0	36
ENE	1	7	11	11	0	0	30
E	0	13	40	37	20	0	110
ESE	4	9	23	17	1	0	54
SE	3	1	3	0	0	0	7
SSE	2	3	1	1	0	0	7
S	3	1	13	3	0	0	20
SSW	1	5	9	5	4	0	24
SW	0	7	18	11	6	7	49
WSW	1	4	7	7	10	3	32
W	3	10	28	33	11	6	91
WNW	0	1	14	29	6	4	54
NW	0	3	12	45	17	0	77
NNW	0	1	3	17	6	0	27
N	0	1	2	4	3	0	10
TOTAL	18	73	213	221	85	20	630

PERIODS OF CALM (NO. OF HOURS) – 0

MISSING DATA (NO. OF HOURS) – 114

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) MAR 1975

TEMP. LAPSE RATE STABILITY CLASS A
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	0	0	0	0	0	0
NE	0	0	0	0	0	0	0
ENE	0	0	0	0	0	0	0
E	0	0	0	0	0	0	0
ESE	0	0	0	0	0	0	0
SE	0	0	0	0	0	0	0
SSE	0	0	0	0	0	0	0
S	0	0	0	0	0	0	0
SSW	0	0	0	0	0	0	0
SW	0	0	0	0	0	0	0
WSW	0	0	0	0	0	0	0
W	0	0	0	0	0	0	0
WNW	0	0	0	0	0	0	0
NW	0	0	0	0	0	0	0
NNW	0	0	0	0	0	0	0
N	0	0	0	0	0	0	0
TOTAL	0	0	0	0	0	0	0

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) MAR 1975

TEMP.LAPSE RATE STABILITY CLASS B
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	0	0	0	0	0	0
NE	0	0	0	0	0	0	0
ENE	0	0	0	0	0	0	0
E	0	0	0	0	0	0	0
ESE	0	0	0	0	0	0	0
SE	0	0	0	0	0	0	0
SSE	0	0	0	0	0	0	0
S	0	0	0	0	0	0	0
SSW	0	0	0	0	0	0	0
SW	0	0	0	0	0	0	0
WSW	0	0	0	0	0	0	0
W	0	0	0	0	0	0	0
WNW	0	0	0	0	0	0	0
NW	0	0	0	0	0	0	0
NNW	0	0	0	0	0	0	0
N	0	0	0	0	0	0	0
TOTAL	0	0	0	0	0	0	0

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) MAR 1975

TEMP. LAPSE RATE STABILITY CLASS C
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 – METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	0	0	0	0	0	0
NE	0	0	0	0	0	0	0
ENE	0	0	0	0	0	0	0
E	0	0	0	0	1	0	1
ESE	0	0	0	0	0	0	0
SE	0	0	0	0	0	0	0
SSE	0	0	0	0	0	0	0
S	0	0	0	0	0	0	0
SSW	0	0	0	0	0	0	0
SW	0	0	0	0	1	0	1
WSW	0	0	0	0	0	0	0
W	0	0	0	0	0	0	0
WNW	0	0	0	0	1	0	1
NW	0	0	0	0	0	0	0
NNW	0	0	0	0	0	0	0
N	0	0	0	0	0	0	0
TOTAL	0	0	0	0	3	0	3

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) MAR 1975

TEMP. LAPSE RATE STABILITY CLASS D
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	1	4	1	1	0	7
NE	0	2	13	0	0	0	15
ENE	0	1	9	9	0	0	19
E	0	6	25	31	18	0	80
ESE	1	2	4	10	1	0	18
SE	2	1	0	0	0	0	3
SSE	1	0	1	0	0	0	2
S	0	0	7	1	0	0	8
SSW	0	0	1	1	2	0	4
SW	0	2	8	7	5	7	29
WSW	0	2	6	5	10	3	26
W	0	1	19	29	10	6	65
WNW	0	0	10	25	5	4	44
NW	0	3	5	36	14	0	58
NNW	0	0	3	16	3	0	22
N	0	0	1	4	3	0	8
TOTAL	4	21	116	175	72	20	408

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) MAR 1975

TEMP. LAPSE RATE STABILITY CLASS E
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	0	1	0	0	0	1
NE	0	4	5	0	0	0	9
ENE	1	5	2	2	0	0	10
E	0	3	13	5	1	0	22
ESE	2	4	9	5	0	0	20
SE	0	0	2	0	0	0	2
SSE	1	2	0	1	0	0	4
S	1	1	4	2	0	0	8
SSW	1	4	8	4	2	0	19
SW	0	5	10	4	0	0	19
WSW	1	2	1	2	0	0	6
W	1	8	9	4	1	0	23
WNW	0	0	4	4	0	0	8
NW	0	0	7	9	3	0	19
NNW	0	1	0	1	3	0	5
N	0	1	1	0	0	0	2
TOTAL	8	40	76	43	10	0	177

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) MAR 1975

TEMP. LAPSE RATE STABILITY CLASS F
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	0	4	0	0	0	4
NE	0	0	2	0	0	0	2
ENE	0	0	0	0	0	0	0
E	0	1	0	1	0	0	2
ESE	1	2	5	2	0	0	10
SE	0	0	0	0	0	0	0
SSE	0	0	0	0	0	0	0
S	2	0	2	0	0	0	4
SSW	0	1	0	0	0	0	1
SW	0	0	0	0	0	0	0
WSW	0	0	0	0	0	0	0
W	2	1	0	0	0	0	3
WNW	0	1	0	0	0	0	1
NW	0	0	0	0	0	0	0
NNW	0	0	0	0	0	0	0
N	0	0	0	0	0	0	0
TOTAL	5	6	13	3	0	0	27

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) MAR 1975

TEMP. LAPSE RATE STABILITY CLASS G
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	0	0	0	0	0	0
NE	0	0	0	0	0	0	0
ENE	0	1	0	0	0	0	1
E	0	3	2	0	0	0	5
ESE	0	1	5	0	0	0	6
SE	1	0	1	0	0	0	2
SSE	0	1	0	0	0	0	1
S	0	0	0	0	0	0	0
SSW	0	0	0	0	0	0	0
SW	0	0	0	0	0	0	0
WSW	0	0	0	0	0	0	0
W	0	0	0	0	0	0	0
WNW	0	0	0	0	0	0	0
NW	0	0	0	0	0	0	0
NNW	0	0	0	0	0	0	0
N	0	0	0	0	0	0	0
TOTAL	1	6	8	0	0	0	15

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

24 Hour Summary of Wind Speed Distribution

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) APR 1975

TOTAL NUMBER OF READINGS	6.70000E+02
TOTAL NUMBER OF READINGS WITHOUT CALMS	6.70000E+02

****

WIND SPEED DISTRIBUTION, NO OF OBS.

LT .5	.50 – 3.5	3.51 – 7.5	7.51 – 12.5	12.51 – 18.5	18.51 – 24.0	GT 24.0
0	19	134	282	145	49	41

****

SUMMED OVER ALL DIRECTIONS
WIND SPEED DISTRIBUTION VERSUS TEMP. LAPSE RATE STABILITY CLASS (NO OF OBS.)

	A	B	C	D	E	F	G
1	0	0	0	0	0	0	0
2	0	0	0	3	5	7	4
3	0	0	0	78	35	15	6
4	0	0	0	229	40	7	6
5	0	0	1	117	23	4	0
6	0	1	3	36	9	0	0
7	0	0	1	38	2	0	0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) APR 1975

****

SUMMED OVER ALL TEMP. LAPSE RATE STABILITIES
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	1	11	26	4	0	0	42
NE	0	21	42	26	1	2	92
ENE	1	13	40	7	6	2	69
E	0	20	51	22	1	0	94
ESE	1	11	33	4	0	0	49
SE	1	8	15	4	0	0	28
SSE	0	6	7	1	0	0	14
S	0	5	1	1	2	0	9
SSW	1	3	5	9	7	1	26
SW	3	5	4	0	3	2	17
WSW	2	10	1	13	5	12	43
W	5	5	2	5	3	0	20
WNW	0	2	7	8	1	2	20
NW	2	4	16	6	13	20	61
NNW	0	3	15	34	3	0	55
N	2	7	17	1	4	0	31
TOTAL	19	134	282	145	49	41	670

PERIODS OF CALM (NO. OF HOURS) – 0

MISSING DATA (NO. OF HOURS) – 50

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) APR 1975

TEMP. LAPSE RATE STABILITY CLASS A
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	0	0	0	0	0	0
NE	0	0	0	0	0	0	0
ENE	0	0	0	0	0	0	0
E	0	0	0	0	0	0	0
ESE	0	0	0	0	0	0	0
SE	0	0	0	0	0	0	0
SSE	0	0	0	0	0	0	0
S	0	0	0	0	0	0	0
SSW	0	0	0	0	0	0	0
SW	0	0	0	0	0	0	0
WSW	0	0	0	0	0	0	0
W	0	0	0	0	0	0	0
WNW	0	0	0	0	0	0	0
NW	0	0	0	0	0	0	0
NNW	0	0	0	0	0	0	0
N	0	0	0	0	0	0	0
TOTAL	0	0	0	0	0	0	0

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) APR 1975

TEMP. LAPSE RATE STABILITY CLASS B
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	0	0	0	0	0	0
NE	0	0	0	0	0	0	0
ENE	0	0	0	0	0	0	0
E	0	0	0	0	0	0	0
ESE	0	0	0	0	0	0	0
SE	0	0	0	0	0	0	0
SSE	0	0	0	0	0	0	0
S	0	0	0	0	0	0	0
SSW	0	0	0	0	0	0	0
SW	0	0	0	0	0	0	0
WSW	0	0	0	0	0	0	0
W	0	0	0	0	0	0	0
WNW	0	0	0	0	0	0	0
NW	0	0	0	0	1	0	1
NNW	0	0	0	0	0	0	0
N	0	0	0	0	0	0	0
TOTAL	0	0	0	0	1	0	1

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) APR 1975

TEMP. LAPSE RATE STABILITY CLASS C
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	0	0	0	0	0	0
NE	0	0	0	0	0	0	0
ENE	0	0	0	0	0	0	0
E	0	0	0	0	0	0	0
ESE	0	0	0	0	0	0	0
SE	0	0	0	0	0	0	0
SSE	0	0	0	0	0	0	0
S	0	0	0	0	0	0	0
SSW	0	0	0	0	0	0	0
SW	0	0	0	0	0	0	0
WSW	0	0	0	0	0	0	0
W	0	0	0	0	0	0	0
WNW	0	0	0	0	0	0	0
NW	0	0	0	1	3	1	5
NNW	0	0	0	0	0	0	0
N	0	0	0	0	0	0	0
TOTAL	0	0	0	1	3	1	5

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) APR 1975

TEMP. LAPSE RATE STABILITY CLASS D
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	1	11	26	4	0	0	42
NE	0	18	40	24	1	2	85
ENE	0	10	37	7	6	2	62
E	0	15	40	17	0	0	72
ESE	0	7	22	3	0	0	32
SE	0	4	10	1	0	0	15
SSE	0	2	2	1	0	0	5
S	0	1	1	0	0	0	2
SSW	1	0	0	6	4	1	12
SW	0	1	1	0	2	0	4
WSW	0	1	1	9	4	12	27
W	0	2	1	5	2	0	10
WNW	0	0	5	4	1	2	12
NW	0	1	14	2	9	19	45
NNW	0	0	15	33	3	0	51
N	1	5	14	1	4	0	25
TOTAL	3	78	229	117	36	38	501

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) APR 1975

TEMP. LAPSE RATE STABILITY CLASS E
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	0	0	0	0	0	0
NE	0	2	2	2	0	0	6
ENE	1	2	1	0	0	0	4
E	0	4	8	1	1	0	14
ESE	0	1	5	1	0	0	7
SE	1	3	4	3	0	0	11
SSE	0	4	5	0	0	0	9
S	0	3	0	1	2	0	6
SSW	0	2	5	3	3	0	13
SW	1	2	3	0	1	2	9
WSW	0	3	0	4	1	0	8
W	1	0	0	0	1	0	2
WNW	0	1	2	4	0	0	7
NW	0	3	2	3	0	0	8
NNW	0	3	0	1	0	0	4
N	1	2	3	0	0	0	6
TOTAL	5	35	40	23	9	2	114

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) APR 1975

TEMP. LAPSE RATE STABILITY CLASS F
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	0	0	0	0	0	0
NE	0	1	0	0	0	0	1
ENE	0	0	2	0	0	0	2
E	0	0	0	4	0	0	4
ESE	1	0	4	0	0	0	5
SE	0	1	0	0	0	0	1
SSE	0	0	0	0	0	0	0
S	0	1	0	0	0	0	1
SSW	0	1	0	0	0	0	1
SW	2	2	0	0	0	0	4
WSW	0	5	0	0	0	0	5
W	2	3	1	0	0	0	6
WNW	0	1	0	0	0	0	1
NW	2	0	0	0	0	0	2
NNW	0	0	0	0	0	0	0
N	0	0	0	0	0	0	0
TOTAL	7	15	7	4	0	0	33

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) APR 1975

TEMP. LAPSE RATE STABILITY CLASS G
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	0	0	0	0	0	0
NE	0	0	0	0	0	0	0
ENE	0	1	0	0	0	0	1
E	0	1	3	0	0	0	4
ESE	0	3	2	0	0	0	5
SE	0	0	0	1	0	0	1
SSE	0	0	0	0	0	0	0
S	0	0	0	0	0	0	0
SSW	0	0	0	0	0	0	0
SW	0	0	0	0	0	0	0
WSW	2	1	0	0	0	0	3
W	2	0	0	0	0	0	2
WNW	0	0	0	0	0	0	0
NW	0	0	0	0	0	0	0
NNW	0	0	0	0	0	0	0
N	0	0	0	0	0	0	0
TOTAL	4	6	6	0	0	0	16

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ∆T

24 – Hour Summary of Wind Speed Distribution

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) MAY 1975

TOTAL NUMBER OF READINGS	7.38000E+02
TOTAL NUMBER OF READINGS WITHOUT CALMS	7.35000E+02

****

WIND SPEED DISTRIBUTION, NO OF OBS.

LT .5	.50 – 3.5	3.51 – 7.5	7.51 – 12.5	12.51 – 18.5	18.51 – 24.0	GT 24.0
3	49	290	292	104	0	0

****

SUMMED OVER ALL DIRECTIONS
WIND SPEED DISTRIBUTION VERSUS TEMP.LAPSE RATE STABILITY CLASS (NO OF OBS.)

	A	B	C	D	E	F	G
1	0	0	0	0	2	1	0
2	0	0	0	15	32	1	1
3	0	2	7	117	101	45	18
4	0	6	20	163	93	9	1
5	4	8	8	77	7	0	0
6	0	0	2	3	0	0	0
7	0	0	0	1	0	0	0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) MAY 1975

****

SUMMED OVER ALL TEMP. LAPSE RATE STABILITIES
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	20	19	2	0	0	41
NE	0	21	22	2	0	0	45
ENE	2	15	32	1	0	0	50
E	4	34	43	11	0	0	92
ESE	4	13	21	3	0	0	41
SE	3	17	13	1	0	0	34
SSE	5	24	5	1	0	0	35
S	8	24	7	0	0	0	39
SSW	1	25	22	4	0	0	52
SW	5	31	17	17	0	0	70
WSW	2	16	24	8	0	0	50
W	2	9	14	13	0	0	38
WNW	3	8	5	5	0	0	21
NW	3	15	21	23	0	0	62
NNW	4	12	10	8	0	0	34
N	3	6	17	5	0	0	31
TOTAL	49	290	292	104	0	0	735

PERIODS OF CALM (NO. OF HOURS) – 3

MISSING DATA (NO. OF HOURS) – 6

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) MAY 1975

TEMP. LAPSE RATE STABILITY CLASS A
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND DIRECTION	WIND SPEED (MPH) AT 10 METER LEVEL 1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	0	0	0	0	0	0
NE	0	0	0	1	0	0	1
ENE	0	0	0	0	0	0	0
E	0	0	0	2	0	0	2
ESE	0	0	0	0	0	0	0
SE	0	0	0	0	0	0	0
SSE	0	0	0	0	0	0	0
S	0	0	0	0	0	0	0
SSW	0	0	0	1	0	0	1
SW	0	0	0	0	0	0	0
WSW	0	0	0	0	0	0	0
W	0	0	0	0	0	0	0
WNW	0	0	0	0	0	0	0
NW	0	0	0	0	0	0	0
NNW	0	0	0	0	0	0	0
N	0	0	0	0	0	0	0
TOTAL	0	0	0	4	0	0	4

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) MAY 1975

TEMP. LAPSE RATE STABILITY CLASS B
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	0	1	0	0	0	1
NE	0	0	0	0	0	0	0
ENE	0	0	1	0	0	0	1
E	0	0	2	4	0	0	6
ESE	0	0	0	0	0	0	0
SE	0	0	0	0	0	0	0
SSE	0	0	0	0	0	0	0
S	0	0	0	0	0	0	0
SSW	0	0	0	0	0	0	0
SW	0	1	0	4	0	0	5
WSW	0	1	1	0	0	0	2
W	0	0	1	0	0	0	1
WNW	0	0	0	0	0	0	0
NW	0	0	0	0	0	0	0
NNW	0	0	0	0	0	0	0
N	0	0	0	0	0	0	0
TOTAL	0	2	6	8	0	0	16

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) MAY 1975

TEMP. LAPSE RATE STABILITY CLASS C
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	0	1	0	0	0	1
NE	0	1	0	0	0	0	1
ENE	0	0	5	0	0	0	5
E	0	0	2	1	0	0	3
ESE	0	0	0	1	0	0	1
SE	0	0	0	0	0	0	0
SSE	0	0	0	0	0	0	0
S	0	1	0	0	0	0	1
SSW	0	1	1	0	0	0	2
SW	0	1	1	4	0	0	6
WSW	0	3	8	1	0	0	12
W	0	0	2	1	0	0	3
WNW	0	0	0	0	0	0	0
NW	0	0	0	0	0	0	0
NNW	0	0	0	0	0	0	0
N	0	0	0	0	0	0	0
TOTAL	0	7	20	8	0	0	35

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) MAY 1975

TEMP. LAPSE RATE STABILITY CLASS D
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	16	9	2	0	0	27
NE	0	16	10	1	0	0	27
ENE	2	9	20	1	0	0	32
E	2	25	30	3	0	0	60
ESE	1	8	14	2	0	0	25
SE	2	6	3	0	0	0	11
SSE	1	2	2	1	0	0	6
S	1	2	5	0	0	0	8
SSW	0	1	10	3	0	0	14
SW	3	4	11	8	0	0	26
WSW	1	8	13	7	0	0	29
W	0	1	5	12	0	0	18
WNW	0	2	1	5	0	0	8
NW	0	10	10	19	0	0	39
NNW	0	4	8	8	0	0	20
N	2	3	12	5	0	0	22
TOTAL	15	117	163	77	0	0	312

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) MAY 1975

TEMP. LAPSE RATE STABILITY CLASS E
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	4	7	0	0	0	11
NE	0	3	9	0	0	0	12
ENE	0	4	6	0	0	0	10
E	2	7	7	1	0	0	17
ESE	3	3	5	0	0	0	11
SE	1	9	10	1	0	0	21
SSE	4	16	3	0	0	0	23
S	6	10	2	0	0	0	18
SSW	1	8	11	0	0	0	20
SW	2	10	5	1	0	0	18
WSW	1	3	2	0	0	0	6
W	1	3	5	0	0	0	9
WNW	3	6	3	0	0	0	12
NW	3	5	11	4	0	0	23
NNW	4	7	2	0	0	0	13
N	1	3	5	0	0	0	9
TOTAL	32	101	93	7	0	0	233

PERIODS OF CALM (NO. OF HOURS) – 2

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) MAY 1975

TEMP. LAPSE RATE STABILITY CLASS F
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	0	1	0	0	0	1
NE	0	1	3	0	0	0	4
ENE	0	2	0	0	0	0	2
E	0	2	1	0	0	0	3
ESE	0	2	2	0	0	0	4
SE	0	2	0	0	0	0	2
SSE	0	3	0	0	0	0	3
S	1	6	0	0	0	0	7
SSW	0	10	0	0	0	0	10
SW	0	11	0	0	0	0	11
WSW	0	0	0	0	0	0	0
W	0	5	1	0	0	0	6
WNW	0	0	1	0	0	0	1
NW	0	0	0	0	0	0	0
NNW	0	1	0	0	0	0	1
N	0	0	0	0	0	0	0
TOTAL	1	45	9	0	0	0	55

PERIODS OF CALM (NO. OF HOURS) – 1

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) MAY 1975

TEMP. LAPSE RATE STABILITY CLASS G
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	0	0	0	0	0	0
NE	0	0	0	0	0	0	0
ENE	0	0	0	0	0	0	0
E	0	0	1	0	0	0	1
ESE	0	0	0	0	0	0	0
SE	0	0	0	0	0	0	0
SSE	0	3	0	0	0	0	3
S	0	5	0	0	0	0	5
SSW	0	5	0	0	0	0	5
SW	0	4	0	0	0	0	4
WSW	0	1	0	0	0	0	1
W	1	0	0	0	0	0	1
WNW	0	0	0	0	0	0	0
NW	0	0	0	0	0	0	0
NNW	0	0	0	0	0	0	0
N	0	0	0	0	0	0	0
TOTAL	1	18	1	0	0	0	20

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

24 Hour Summary of Wind Speed Distribution

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) JUNE 1975

TOTAL NUMBER OF READINGS 6.92000E+02

TOTAL NUMBER OF READINGS WITHOUT CALMS 6.92000E+02

****

WIND SPEED DISTRIBUTION, NO OF OBS.

LT .5	.50 – 3.5	3.51 – 7.5	7.51 – 12.5	12.51 – 18.5	18.51 – 24.0	GT 24.0
0	32	200	309	123	26	2

****

SUMMED OVER ALL DIRECTIONS
WIND SPEED DISTRIBUTION VERSUS TEMP. LAPSE RATE STABILITY CLASS (NO OF OBS.)

	A	B	C	D	E	F	G
1	0	0	0	0	0	0	0
2	0	1	0	6	21	2	2
3	2	2	4	94	86	10	2
4	5	14	24	178	86	2	0
5	4	7	12	78	22	0	0
6	3	0	4	19	0	0	0
7	0	0	1	1	0	0	0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) JUNE 1975

****

SUMMED OVER ALL TEMP. LAPSE RATE STABILITIES
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	1	11	13	1	1	0	27
NE	3	24	22	1	1	0	51
ENE	3	31	38	5	1	0	78
E	5	24	38	8	2	0	77
ESE	0	4	9	0	0	0	13
SE	4	6	11	3	0	0	24
SSE	2	9	8	6	0	0	25
S	5	13	10	5	2	0	35
SSW	0	29	33	7	1	0	70
SW	2	18	53	34	4	0	111
WSW	1	14	31	17	4	0	67
W	3	10	15	12	2	0	42
WNW	0	0	8	6	4	2	20
NW	1	3	10	12	4	0	30
NNW	2	1	6	5	0	0	14
N	0	3	4	1	0	0	8
TOTAL	32	200	309	123	26	2	692

PERIODS OF CALM (NO. OF HOURS) – 0

MISSING DATA (NO. OF HOURS) – 28

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) JUNE 1975

TEMP. LAPSE RATE STABILITY CLASS A
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	0	0	0	0	0	0
NE	0	0	0	0	0	0	0
ENE	0	0	0	1	1	0	2
E	0	0	0	2	2	0	4
ESE	0	0	0	0	0	0	0
SE	0	0	0	0	0	0	0
SSE	0	0	0	0	0	0	0
S	0	2	0	0	0	0	2
SSW	0	0	2	0	0	0	2
SW	0	0	2	0	0	0	2
WSW	0	0	1	1	0	0	2
W	0	0	0	0	0	0	0
WNW	0	0	0	0	0	0	0
NW	0	0	0	0	0	0	0
NNW	0	0	0	0	0	0	0
N	0	0	0	0	0	0	0
TOTAL	0	2	5	4	3	0	14

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) JUNE 1975

TEMP. LAPSE RATE STABILITY CLASS B
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	0	0	0	0	0	0
NE	0	0	0	0	0	0	0
ENE	0	1	4	1	0	0	6
E	0	0	6	2	0	0	8
ESE	0	0	1	0	0	0	1
SE	1	0	0	0	0	0	1
SSE	0	0	0	0	0	0	0
S	0	1	0	0	0	0	1
SSW	0	0	0	0	0	0	0
SW	0	0	1	3	0	0	4
WSW	0	0	2	1	0	0	3
W	0	0	0	0	0	0	0
WNW	0	0	0	0	0	0	0
NW	0	0	0	0	0	0	0
NNW	0	0	0	0	0	0	0
N	0	0	0	0	0	0	0
TOTAL	1	2	14	7	0	0	24

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) JUNE 1975

TEMP. LAPSE RATE STABILITY CLASS C
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	0	0	0	0	0	0
NE	0	1	2	0	0	0	3
ENE	0	1	10	0	0	0	11
E	0	2	4	0	0	0	6
ESE	0	0	0	0	0	0	0
SE	0	0	1	1	0	0	2
SSE	0	0	0	0	0	0	0
S	0	0	0	0	0	0	0
SSW	0	0	1	0	0	0	1
SW	0	0	4	2	1	0	7
WSW	0	0	2	2	0	0	4
W	0	0	0	2	0	0	2
WNW	0	0	0	0	0	0	0
NW	0	0	0	0	2	1	3
NNW	0	0	0	5	1	0	6
N	0	0	0	0	0	0	0
TOTAL	0	4	24	12	4	1	45

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) JUNE 1975

TEMP. LAPSE RATE STABILITY CLASS D
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	10	13	1	1	0	25
NE	0	17	20	1	1	0	39
ENE	2	17	23	3	0	0	45
E	1	14	25	4	0	0	44
ESE	0	2	7	0	0	0	9
SE	0	4	6	2	0	0	12
SSE	1	2	4	6	0	0	13
S	1	4	4	4	2	0	15
SSW	0	5	8	4	1	0	18
SW	0	3	27	16	3	0	49
WSW	0	7	12	10	4	0	33
W	1	3	6	9	2	0	21
WNW	0	0	4	5	2	1	12
NW	0	2	9	7	3	0	21
NNW	0	1	6	5	0	0	12
N	0	3	4	1	0	0	8
TOTAL	6	94	178	78	19	1	376

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) JUNE 1975

TEMP. LAPSE RATE STABILITY CLASS E
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	1	1	0	0	0	0	2
NE	3	6	0	0	0	0	9
ENE	1	12	1	0	0	0	14
E	4	8	3	0	0	0	15
ESE	0	2	1	0	0	0	3
SE	3	1	4	0	0	0	8
SSE	1	6	4	0	0	0	11
S	3	5	6	1	0	0	15
SSW	0	21	22	3	0	0	46
SW	0	13	19	13	0	0	45
WSW	1	7	13	3	0	0	24
W	1	3	8	1	0	0	13
WNW	0	0	4	1	0	0	5
NW	1	1	1	0	0	0	3
NNW	2	0	0	0	0	0	2
N	0	0	0	0	0	0	0
TOTAL	21	86	86	22	0	0	215

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) JUNE 1975

TEMP. LAPSE RATE STABILITY CLASS F
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	0	0	0	0	0	0
NE	0	0	0	0	0	0	0
ENE	0	0	0	0	0	0	0
E	0	0	0	0	0	0	0
ESE	0	0	0	0	0	0	0
SE	0	1	0	0	0	0	1
SSE	0	1	0	0	0	0	1
S	1	1	0	0	0	0	2
SSW	0	3	0	0	0	0	3
SW	0	2	0	0	0	0	2
WSW	0	0	1	0	0	0	1
W	1	2	1	0	0	0	4
WNW	0	0	0	0	0	0	0
NW	0	0	0	0	0	0	0
NNW	0	0	0	0	0	0	0
N	0	0	0	0	0	0	0
TOTAL	2	10	2	0	0	0	14

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) JUNE 1975

TEMP. LAPSE RATE STABILITY CLASS G
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	0	0	0	0	0	0
NE	0	0	0	0	0	0	0
ENE	0	0	0	0	0	0	0
E	0	0	0	0	0	0	0
ESE	0	0	0	0	0	0	0
SE	0	0	0	0	0	0	0
SSE	0	0	0	0	0	0	0
S	0	0	0	0	0	0	0
SSW	0	0	0	0	0	0	0
SW	2	0	0	0	0	0	2
WSW	0	0	0	0	0	0	0
W	0	2	0	0	0	0	2
WNW	0	0	0	0	0	0	0
NW	0	0	0	0	0	0	0
NNW	0	0	0	0	0	0	0
N	0	0	0	0	0	0	0
TOTAL	2	2	0	0	0	0	4

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

24 Hour Summary of Wind Speed Distribution

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) JULY 1975

TOTAL NUMBER OF READINGS	7.05000E+02
TOTAL NUMBER OF READINGS WITHOUT CALMS	7.05000E+02

****

WIND SPEED DISTRIBUTION, NO OF OBS.

LT .5	.50 – 3.5	3.51 – 7.5	7.51 – 12.5	12.51 – 18.5	18.51 – 24.0	GT 24.0
0	115	302	214	67	7	0

****

SUMMED OVER ALL DIRECTIONS
WIND SPEED DISTRIBUTION VERSUS TEMP. LAPSE RATE STABILITY CLASS (NO OF OBS.)

	A	B	C	D	E	F	G
1	0	0	0	0	0	0	0
2	1	8	7	14	28	31	26
3	10	30	27	61	93	62	19
4	5	27	28	98	47	9	0
5	0	8	7	44	8	0	0
6	0	1	1	4	1	0	0
7	0	0	0	0	0	0	0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) JULY 1975

****

SUMMED OVER ALL TEMP. LAPSE RATE STABILITIES
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	3	9	1	0	0	13
NE	4	16	10	2	0	0	32
ENE	3	28	26	1	0	0	58
E	5	21	16	0	0	0	42
ESE	1	7	4	0	0	0	12
SE	9	18	8	0	0	0	35
SSE	14	26	2	0	0	0	42
S	15	33	10	1	0	0	59
SSW	13	41	21	5	0	0	80
SW	16	34	19	24	2	0	95
WSW	10	21	16	17	4	0	68
W	6	27	14	6	1	0	54
WNW	4	7	13	5	0	0	29
NW	8	3	12	3	0	3	26
NNW	3	6	19	2	0	0	30
N	4	11	15	0	0	0	30
TOTAL	115	302	214	67	7	0	705

PERIODS OF CALM (NO. OF HOURS) – 0

MISSING DATA (NO. OF HOURS) – 39

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) JULY 1975

TEMP. LAPSE RATE STABILITY CLASS A
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	0	0	0	0	0	0
NE	0	1	0	0	0	0	1
ENE	0	3	1	0	0	0	4
E	0	1	0	0	0	0	1
ESE	0	2	0	0	0	0	2
SE	0	1	0	0	0	0	1
SSE	0	0	0	0	0	0	0
S	0	0	2	0	0	0	2
SSW	0	2	1	0	0	0	3
SW	0	0	0	0	0	0	0
WSW	0	0	0	0	0	0	0
W	0	0	0	0	0	0	0
WNW	0	0	0	0	0	0	0
NW	0	0	1	0	0	0	1
NNW	0	0	0	0	0	0	0
N	1	0	0	0	0	0	1
TOTAL	1	10	5	0	0	0	16

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) JULY 1975

TEMP. LAPSE RATE STABILITY CLASS B
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	0	0	0	0	0	0
NE	0	6	3	0	0	0	9
ENE	1	6	9	0	0	0	16
E	3	3	1	0	0	0	7
ESE	0	0	0	0	0	0	0
SE	0	0	1	0	0	0	1
SSE	0	2	1	0	0	0	3
S	1	3	0	0	0	0	4
SSW	1	3	2	1	0	0	7
SW	0	1	0	4	0	0	5
WSW	1	2	1	2	1	0	7
W	0	0	0	0	0	0	0
WNW	0	1	5	0	0	0	6
NW	0	1	3	1	0	0	5
NNW	0	0	1	0	0	0	1
N	1	2	0	0	0	0	3
TOTAL	8	30	27	8	1	0	74

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) JULY 1975

TEMP. LAPSE RATE STABILITY CLASS C
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	2	2	0	0	0	4
NE	2	2	2	0	0	0	6
ENE	0	4	9	0	0	0	13
E	1	5	2	0	0	0	8
ESE	0	1	0	0	0	0	1
SE	2	0	0	0	0	0	2
SSE	0	2	1	0	0	0	3
S	1	1	1	0	0	0	3
SSW	0	1	0	1	0	0	2
SW	0	3	0	3	0	0	6
WSW	0	2	1	3	1	0	7
W	0	0	0	0	0	0	0
WNW	0	2	2	0	0	0	4
NW	1	0	2	0	0	0	3
NNW	0	0	6	0	0	0	6
N	0	2	0	0	0	0	2
TOTAL	7	27	28	7	1	0	70

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) JULY 1975

TEMP. LAPSE RATE STABILITY CLASS D
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND DIRECTION	WIND SPEED (MPH) AT 10 METER LEVEL 1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	1	7	1	0	0	9
NE	2	4	4	2	0	0	12
ENE	1	9	4	1	0	0	15
E	0	6	12	0	0	0	18
ESE	1	2	1	0	0	0	4
SE	3	1	2	0	0	0	6
SSE	0	1	0	0	0	0	1
S	1	5	3	1	0	0	10
SSW	1	6	6	1	0	0	14
SW	2	10	8	15	1	0	36
WSW	1	1	12	12	2	0	28
W	0	3	8	4	1	0	16
WNW	0	0	3	3	0	0	6
NW	1	2	2	2	0	0	7
NNW	1	3	12	2	0	0	18
N	0	7	14	0	0	0	21
TOTAL	14	61	98	44	4	0	221

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) JULY 1975

TEMP. LAPSE RATE STABILITY CLASS E
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	0	0	0	0	0	0
NE	0	3	1	0	0	0	4
ENE	1	5	3	0	0	0	9
E	1	6	1	0	0	0	8
ESE	0	2	3	0	0	0	5
SE	4	14	5	0	0	0	24
SSE	4	16	0	0	0	0	20
S	1	12	4	0	0	0	17
SSW	3	14	11	2	0	0	30
SW	3	8	8	2	1	0	22
WSW	0	2	2	0	0	0	4
W	2	7	1	2	0	0	12
WNW	3	2	3	2	0	0	10
NW	3	0	4	0	0	0	7
NNW	1	2	0	0	0	0	3
N	2	0	1	0	0	0	3
TOTAL	28	93	47	8	1	0	177

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) JULY 1975

TEMP. LAPSE RATE STABILITY CLASS F
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	0	0	0	0	0	0
NE	0	0	0	0	0	0	0
ENE	0	1	0	0	0	0	1
E	0	0	0	0	0	0	0
ESE	0	0	0	0	0	0	0
SE	0	2	0	0	0	0	2
SSE	7	4	0	0	0	0	11
S	3	6	0	0	0	0	9
SSW	2	8	1	0	0	0	11
SW	8	10	3	0	0	0	21
WSW	2	12	0	0	0	0	14
W	4	16	5	0	0	0	25
WNW	1	2	0	0	0	0	3
NW	3	0	0	0	0	0	3
NNW	1	1	0	0	0	0	2
N	0	0	0	0	0	0	0
TOTAL	31	62	9	0	0	0	102

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) JULY 1975

TEMP. LAPSE RATE STABILITY CLASS G
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	0	0	0	0	0	0
NE	0	0	0	0	0	0	0
ENE	0	0	0	0	0	0	0
E	0	0	0	0	0	0	0
ESE	0	0	0	0	0	0	0
SE	0	0	0	0	0	0	0
SSE	3	1	0	0	0	0	4
S	8	6	0	0	0	0	14
SSW	6	7	0	0	0	0	13
SW	3	2	0	0	0	0	5
WSW	6	2	0	0	0	0	8
W	0	1	0	0	0	0	1
WNW	0	0	0	0	0	0	0
NW	0	0	0	0	0	0	0
NNW	0	0	0	0	0	0	0
N	0	0	0	0	0	0	0
TOTAL	26	19	0	0	0	0	45

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

24 HOUR SUMMARY OF WIND SPEED DISTRIBUTION

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) 8/4/74 – 8/3/75

TOTAL NUMBER OF READINGS 8.15700E+03

TOTAL NUMBER OF READINGS WITHOUT CALMS 8.14700E+03

****

WIND SPEED DISTRIBUTION, NO OF OBS.

LT .5	.50 – 3.5	3.51 – 7.5	7.51 – 12.5	12.51 – 18.5	18.51 – 24.0	GT 24.0
10	636	2438	2924	1572	433	144

****

SUMMED OVER ALL DIRECTIONS
WIND SPEED DISTRIBUTION VERSUS TEMP. LAPSE RATE STABILITY CLASS (NO OF OBS.)

	A	B	C	D	E	F	G
1	0	0	0	0	9	1	0
2	6	10	10	147	249	136	78
3	39	49	74	932	844	362	138
4	23	94	174	1690	836	82	25
5	21	74	63	1150	253	11	0
6	4	9	21	331	67	1	0
7	0	0	2	121	21	0	0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) 8/4/74 – 8/3/75

****

SUMMED OVER ALL TEMP. LAPSE RATE STABILITIES
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	11	100	139	52	17	0	319
NE	21	142	222	81	5	2	473
ENE	22	152	237	41	11	4	467
E	38	199	255	90	38	5	625
ESE	30	117	140	44	2	0	333
SE	49	143	101	15	0	1	309
SSE	67	155	63	21	10	0	316
S	101	260	137	19	7	1	525
SSW	63	372	370	111	28	1	945
SW	79	282	374	246	51	23	1055
WSW	41	157	229	259	91	56	833
W	35	124	194	210	74	20	657
WNW	20	48	97	110	33	10	318
NW	25	49	159	130	44	21	428
NNW	17	52	107	111	13	0	300
N	17	76	110	32	9	0	244
TOTAL	636	2438	2924	1572	433	144	8147

PERIODS OF CALM (NO. OF HOURS) – 10

MISSING DATA (NO. OF HOURS) – 603

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) 8/4/74 – 8/3/75

TEMP. LAPSE RATE STABILITY CLASS A
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	1	0	0	0	0	1
NE	0	1	1	1	0	0	3
ENE	0	3	1	4	1	0	9
E	1	2	3	4	2	0	12
ESE	1	4	0	0	0	0	5
SE	0	3	0	0	0	0	3
SSE	0	2	0	0	0	0	2
S	0	3	4	0	0	0	7
SSW	0	18	6	1	0	0	25
SW	1	0	5	5	0	0	11
WSW	0	2	1	1	0	0	4
W	1	0	0	1	1	0	3
WNW	0	0	0	1	0	0	1
NW	1	0	2	1	0	0	4
NNW	0	0	0	2	0	0	2
N	1	0	0	0	0	0	1
TOTAL	6	39	23	21	4	0	93

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) 8/4/74 – 8/3/75

TEMP. LAPSE RATE STABILITY CLASS B
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	1	1	1	0	0	3
NE	0	8	6	6	0	0	20
ENE	1	10	17	2	0	0	30
E	3	5	12	7	0	0	27
ESE	0	0	1	0	0	0	1
SE	1	0	1	0	0	0	2
SSE	0	2	1	0	0	0	3
S	2	5	3	1	0	0	11
SSW	1	5	11	6	0	0	23
SW	0	3	11	25	4	0	47
WSW	1	4	8	9	2	0	24
W	0	0	4	8	1	0	13
WNW	0	3	8	0	1	0	12
NW	0	1	8	6	1	0	16
NNW	0	0	1	3	0	0	4
N	1	2	1	0	0	0	4
TOTAL	10	49	94	74	9	0	236

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) 8/4/74 – 8/3/75

TEMP. LAPSE RATE STABILITY CLASS C
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	6	3	1	0	0	10
NE	2	7	14	1	0	0	24
ENE	0	6	39	0	0	0	45
E	1	12	14	2	1	0	30
ESE	0	5	1	1	0	0	7
SE	3	2	1	1	0	0	7
SSE	0	2	1	0	0	0	3
S	2	3	2	0	0	0	7
SSW	0	8	19	8	0	0	35
SW	1	7	21	15	5	0	49
WSW	0	7	14	13	6	0	40
W	1	7	6	1	0	0	15
WNW	0	3	8	4	4	1	20
NW	1	1	10	7	4	1	24
NNW	0	1	15	4	0	0	20
N	0	3	5	0	0	0	8
TOTAL	10	74	174	63	21	2	344

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) 8/4/74 – 8/3/75

TEMP. LAPSE RATE STABILITY CLASS D
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	6	84	117	49	17	0	273
NE	6	96	169	70	5	2	348
ENE	12	85	144	32	10	4	287
E	13	121	167	63	31	2	397
ESE	11	58	73	30	2	0	174
SE	10	33	46	6	0	0	95
SSE	10	27	28	12	0	0	77
S	14	50	52	10	2	0	128
SSW	10	74	143	56	19	1	303
SW	14	79	170	147	30	18	458
WSW	9	46	140	193	68	46	502
W	7	43	119	665	64	20	418
WNW	5	13	56	87	28	8	197
NW	7	37	92	99	36	20	291
NNW	5	27	84	99	10	0	225
N	8	59	90	32	9	0	198
TOTAL	147	932	1690	1150	331	121	4371

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) 8/4/74 – 8/3/75

TEMP. LAPSE RATE STABILITY CLASS E
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND DIRECTION	WIND SPEED (MPH) AT 10 METER LEVEL 1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	5	8	13	1	0	0	27
NE	12	28	27	3	0	0	70
ENE	8	43	33	3	0	0	81
E	17	50	45	9	4	3	128
ESE	13	30	42	11	0	0	96
SE	24	72	46	8	0	1	151
SSE	27	81	29	9	9	0	155
S	40	105	74	8	5	1	233
SSW	23	146	175	38	9	0	391
SW	22	110	152	52	12	5	353
WSW	11	57	65	43	15	10	201
W	7	44	56	30	7	0	144
WNW	12	22	23	18	0	1	76
NW	11	17	36	17	3	0	84
NNW	11	20	6	3	3	0	43
N	6	11	14	0	0	0	31
TOTAL	249	844	836	253	67	21	2270

PERIODS OF CALM (NO. OF HOURS) – 9

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) 8/4/74 – 8/3/75

TEMP. LAPSE RATE STABILITY CLASS F
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	0	5	0	0	0	5
NE	0	2	5	0	0	0	7
ENE	1	3	2	0	0	0	6
E	2	4	7	5	0	0	18
ESE	5	12	14	2	0	0	33
SE	8	27	5	0	0	0	40
SSE	20	31	4	0	1	0	56
S	22	61	2	0	0	0	85
SSW	17	87	14	2	0	0	120
SW	29	57	12	2	0	0	100
WSW	7	31	1	0	0	0	39
W	15	33	8	0	0	0	56
WNW	3	7	1	0	0	0	11
NW	5	3	1	0	0	0	9
NNW	1	3	1	0	0	0	5
N	1	1	0	0	0	0	2
TOTAL	136	362	82	11	1	0	592

PERIODS OF CALM (NO. OF HOURS) – 1

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) 8/4/74 – 8/3/75

TEMP. LAPSE RATE STABILITY CLASS G
WIND SPEED VERSUS DIRECTION (IN NUMBER OF OBS.)

WIND	WIND SPEED (MPH) AT 10 METER LEVEL						
DIRECTION	1 – 3	4 – 7	8 – 12	13 – 18	19 – 24	>24	TOTAL
NNE	0	0	0	0	0	0	0
NE	1	0	0	0	0	0	1
ENE	0	2	1	0	0	0	3
E	1	5	7	0	0	0	13
ESE	0	8	9	0	0	0	17
SE	3	6	2	0	0	0	11
SSE	10	10	0	0	0	0	20
S	21	33	0	0	0	0	54
SSW	12	34	2	0	0	0	48
SW	12	26	3	0	0	0	41
WSW	13	10	0	0	0	0	23
W	5	3	0	0	0	0	8
WNW	0	0	1	0	0	0	1
NW	0	0	0	0	0	0	0
NNW	0	1	0	0	0	0	1
N	0	0	0	0	0	0	0
TOTAL	78	138	25	0	0	0	241

PERIODS OF CALM (NO. OF HOURS) – 0

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

** ANNUAL AVERAGE ** **12 MO DATA** TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) 8/4/74 – 8/3/75

HOURLY TEMP. LAPSE RATE STABILITY INDEX DISTRIBUTION TOTAL NO OF OBS = 8157

	IN PERCENT OF TOTAL OBS							*IN PERCENT OF HOURLY OBS*						
HOUR INDEX	1	2	3	4	5	6	7	1	2	3	4	5	6	7
1	.01	.01	0.00	1.66	1.78	.50	.27	.29	.29	0.00	39.13	42.03	11.88	6.38
2	0.00	0.00	.04	1.70	1.64	.50	.34	0.00	0.00	.87	40.29	38.84	11.88	8.12
3	0.00	0.00	.0.00	1.72	1.68	.53	.32	0.00	0.00	0.00	40.46	39.60	12.43	7.51
4	.01	0.00	.01	1.69	1.57	.60	.32	.29	0.00	.29	40.23	37.32	14.29	7.58
5	.02	0.00	0.00	1.75	1.54	.55	.33	.58	0.00	0.00	41.69	36.73	13.12	7.87
6	.01	.02	0.00	1.78	1.53	.55	.27	.29	.59	0.00	42.65	36.76	13.24	6.47
7	.04	0.00	0.00	2.34	1.20	.48	.15	.87	0.00	0.00	55.69	28.57	11.37	3.50
8	.05	.02	.07	2.77	.80	.28	.10	1.20	.60	1.80	67.66	19.46	6.89	2.40
9	.06	.11	.31	2.88	.53	.15	.01	1.52	2.73	7.58	71.21	13.03	3.64	.30
10	.11	.33	.38	2.72	.44	.01	.02	2.74	8.23	9.45	67.68	10.98	.30	.61
11	.12	.50	.50	2.72	.28	.01	0.00	2.96	12.13	12.13	65.68	6.80	.30	0.00
12	.21	.45	.47	2.67	.25	.02	0.00	5.12	11.14	11.45	65.66	6.02	.60	0.00
13	.18	.47	.44	2.73	.27	.01	0.00	4.48	11.34	10.75	66.57	6.57	.30	0.00
14	.10	.39	.58	2.78	.28	.02	0.00	2.36	9.44	13.86	66.96	6.78	.59	0.00
15	.04	.27	.70	2.68	.34	.02	.02	.90	6.61	17.12	65.77	8.41	.60	.60
16	.02	.15	.36	3.11	.50	.01	.02	.59	3.52	8.50	74.49	12.02	.29	.59
17	.04	.10	.20	2.93	.85	.06	.02	.88	2.34	4.68	69.88	20.18	1.46	.58
18	.01	.02	.07	2.73	1.21	.10	.04	.29	.58	1.75	65.20	28.95	2.34	.88
19	.01	.01	.01	2.11	1.70	.32	.02	.29	.29	.29	50.29	40.64	7.63	.58
20	.01	.01	.01	1.69	1.95	.45	.05	.29	.29	.29	40.47	46.63	10.85	1.17
21	.01	.01	.04	1.59	1.92	.53	.10	.29	.29	.87	37.90	45.77	12.54	2.33
22	.02	0.00	0.00	1.61	1.92	.53	.12	.58	0.00	0.00	38.19	45.77	12.54	2.92
23	.02	0.00	0.00	1.62	1.86	.51	.20	.58	0.00	0.00	38.37	44.19	12.21	4.65
24	.01	0.00	.04	1.58	1.88	.50	.22	.29	0.00	.87	37.39	44.35	11.88	5.22

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

** ANNUAL AVERAGE ** **12 MO DATA** TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) 8/4/74 – 8/3/75

HOURLY TEMP. LAPSE RATE STABILITY INDEX DISTRIBUTION TOTAL NO OF OBS = 8157

TEMPERATURE LAPSE RATE STABILITY INDEX DISTRIBUTION (IN PERCENT OF TOTAL OBS)

INDEX	1	2	3	4	5	6	7
	1.14	2.89	4.22	53.59	27.94	7.27	2.95

AVERAGE WIND SPEED FOR EACH TEMPERATURE LAPSE RATE STABILITY INDEX (IN MPH)

INDEX	1	2	3	4	5	6	7
SPEED	9.4	11.0	10.8	11.8	8.6	5.5	4.9

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

** ANNUAL AVERAGE ** **12 MO DATA** TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) 8/4/74 – 8/3/75

WIND ROSE FOR EACH TEMP. LAPSE RATE STABILITY INDEX (IN PERCENT OF EACH INDEX TOTAL)

INDEX	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	N	CALM
1	1.08	3.23	9.68	12.90	5.38	3.23	2.15	7.53	26.88	11.83	4.30	3.23	1.68	4.30	2.15	1.08	0.00
2	1.27	8.47	12.71	11.44	.42	.85	1.27	4.66	9.75	18.22	10.17	5.51	5.08	6.78	1.69	1.69	0.00
3	2.91	6.98	13.08	8.72	2.03	2.03	.87	2.03	10.17	14.24	11.63	4.36	5.81	6.98	5.81	2.33	0.00
4	6.25	7.96	6.57	9.08	3.98	2.17	1.76	2.93	6.93	10.48	11.48	9.56	4.51	6.66	5.15	4.53	0.00
5	1.18	3.07	3.82	5.62	4.21	6.63	6.80	10.22	17.16	15.49	8.82	6.32	3.33	3.69	1.89	1.36	.39
6	.84	1.18	1.01	3.04	5.56	6.75	9.44	14.33	20.24	16.86	6.58	9.44	1.85	1.52	.84	.34	.17
7	0.00	.41	1.24	5.39	7.05	4.56	8.30	22.41	19.92	17.01	9.54	3.32	.41	0.00	.41	0.00	0.00

** TOTAL NO OF OBS = 8157 **

** GROSS WIND ROSE (IN PERCENT OF TOTAL OBS) **

	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	N	CALM
	3.91	5.80	5.73	7.66	4.08	3.79	3.87	6.44	11.59	12.93	10.21	8.05	3.90	5.25	3.68	2.99	.12
SPEED	10.1	9.5	9.2	9.9	8.5	7.0	6.9	6.6	8.8	10.6	13.1	12.4	12.6	12.9	11.5	9.3	.3

**TEMP. LAPSE RATE STABILITY INDEX DISTRIBUTION FOR EACH WIND DIRECTION (IN PERCENT OF DIRECTION TOTAL) **

INDEX	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	N	CALM
1	.31	.63	1.93	1.92	1.50	.97	.63	1.33	2.65	1.04	.48	.46	.31	.93	.67	.41	0.00
2	.94	4.23	6.42	4.32	.30	.65	.95	2.10	2.43	4.08	2.88	1.98	3.77	3.74	1.33	1.64	0.00
3	3.13	5.07	9.64	4.80	2.10	2.27	.95	1.33	3.70	4.64	4.80	2.28	6.29	5.61	6.67	3.28	0.00
4	85.58	73.57	61.46	63.52	52.25	30.74	24.37	24.38	32.06	43.41	60.26	63.62	61.95	67.99	75.00	81.15	0.00
5	8.46	14.80	18.63	20.48	28.83	48.87	49.05	44.38	41.38	33.46	24.13	21.92	23.90	19.63	14.33	12.70	90.00
6	1.57	1.48	1.28	2.88	9.91	12.94	17.72	16.19	12.70	9.48	4.68	8.52	3.46	2.10	1.67	.82	10.00
7	0.00	.21	.64	2.08	5.11	3.56	6.33	10.29	5.08	3.89	2.76	1.22	.31	0.00	.33	0.00	0.00

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

** ANNUAL AVERAGE ** **12 MO DATA** TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) 8/4/74 – 8/3/75

Temp. Lapse Rate Stability Index Distribution in Percent of Total Obs

Index	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	N	Calm
1	.01	.04	.11	.15	.06	.04	.02	.09	.31	.13	.05	.04	.01	.05	.02	.01	0.00
2	.04	.25	.37	.33	.01	.02	.04	.13	.28	.53	.29	.16	.15	.20	.05	.05	0.00
3	.12	.29	.55	.37	.09	.09	.04	.09	.43	.60	.49	.18	.25	.29	.25	.10	0.00
4	3.35	4.27	3.52	4.87	2.13	1.16	.94	1.57	3.71	5.61	6.15	5.12	2.42	3.57	2.76	2.43	0.00
5	.33	.86	1.07	1.57	1.18	1.85	1.90	2.86	4.79	4.33	2.46	1.77	.93	1.03	.53	.38	.11
6	.06	.09	.07	.22	.40	.49	.69	1.04	1.47	1.23	.48	.69	.13	.11	.06	.02	.01
7	0.00	.01	.04	.16	.21	.13	.25	.66	.59	.50	.28	.10	.01	0.00	.01	0.00	0.00

** Average Wind Speed (Inverse Weighted) by Index and Direction (in MPH)**

Index	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	N
1	7.00	7.91	10.84	4.45	3.86	6.80	4.63	6.63	6.69	7.40	7.97	3.88	15.50	6.13	15.15	3.00
2	8.35	8.34	7.52	7.71	8.50	4.36	6.33	5.62	8.35	12.78	9.35	14.05	9.91	11.45	13.26	4.54
3	7.07	7.07	9.43	7.76	7.75	4.28	5.67	5.56	9.51	9.66	11.20	11.73	11.03	8.59	9.88	8.17
4	8.50	8.60	8.07	8.09	7.09	6.68	6.55	6.12	8.67	9.79	11.80	11.68	11.26	10.78	10.16	8.16
5	4.84	1.52	5.42	5.95	6.52	5.30	5.28	5.10	7.07	7.37	8.06	7.50	6.09	6.63	4.20	4.87
6	10.45	7.75	5.96	7.40	4.99	4.38	3.67	3.75	4.96	4.27	4.01	4.11	4.40	3.96	4.61	2.23
7	0.00	1.00	6.25	7.18	7.16	4.50	3.55	3.64	4.06	4.23	2.82	2.56	8.00	0.00	5.50	0.00

(Average Inverse Speed)

Index	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	N
1	.14	.13	.09	.22	.26	.15	.22	.15	.15	.14	.13	.26	.06	.16	.07	.33
2	.12	.12	.13	.13	.12	.23	.15	.18	.12	.08	.11	.07	.10	.09	.08	.22
3	.14	.14	.11	.13	.13	.23	.18	.18	.11	.10	.09	.09	.09	.12	.10	.12
4	.12	.12	.12	.12	.14	.15	.15	.16	.12	.10	.08	.09	.09	.09	.10	.12
5	.21	.18	.18	.17	.15	.19	.19	.20	.14	.14	.12	.13	.16	.15	.24	.21
6	.10	.13	.17	.14	.20	.23	.27	.27	.20	.23	.25	.24	.23	.25	.22	.45
7	0.00	1.00	.16	.14	.14	.22	.28	.27	.25	.24	.35	.39	.13	0.00	.18	0.00

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

** ANNUAL AVERAGE ** **12 MO DATA** TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) 8/4/74 – 8/3/75
DIRECTIONS INDICATED ARE SECTORS FROM WHICH THE WIND IS BLOWING

**CHI/Q FOR RELEASE HEIGHT OF * 0. METERS * (IN SEC PER CU METER) **

Dist, M	NNE	NE	ENE	E	ESE	SE	SSE	S
6.0500E+02	1.0550E-06	1.6797E-06	1.6771E-06	2.3808E-06	1.7140E-06	2.0811E-06	2.6448E-06	4.6754E-06
6.2000E+02	1.0196E-06	1.6202E-06	1.6161E-06	2.2995E-06	1.6488E-06	1.9976E-06	2.5373E-06	4.4857E-06
6.3500E+02	9.8601E-07	1.5641E-06	1.5587E-06	2.2173E-06	1.5874E-06	1.9192E-06	2.4365E-06	4.3079E-06
6.4000E+02	9.7522E-07	1.5461E-06	1.5402E-06	2.1909E-06	1.5678E-06	1.8942E-06	2.4044E-06	4.2511E-06
6.7000E+02	9.1429E-07	1.4446E-06	1.4367E-06	2.0428E-06	1.4578E-06	1.7544E-06	2.2247E-06	3.9339E-06
6.8000E+02	8.9534E-07	1.4132E-06	1.4046E-06	1.9970E-06	1.4239E-06	1.7114E-06	2.1695E-06	3.8365E-06
7.2000E+02	8.2547E-07	1.2978E-06	1.2872E-06	1.8293E-06	1.3000E-06	1.5550E-06	1.9690E-06	3.4825E-06
7.2500E+02	8.1736E-07	1.2844E-06	1.2736E-06	1.8099E-06	1.2858E-06	1.5371E-06	1.9460E-06	3.4419E-06
7.3000E+02	8.0937E-07	1.2713E-06	1.2603E-06	1.7909E-06	1.2718E-06	1.5194E-06	1.9234E-06	3.4021E-06
7.4000E+02	7.9376E-07	1.2456E-06	1.2343E-06	1.7537E-06	1.2444E-06	1.4852E-06	1.8795E-06	3.3245E-06
8.1500E+02	6.9053E-07	1.0770E-06	1.0636E-06	1.5105E-06	1.0664E-06	1.2629E-06	1.5953E-06	2.8226E-06
8.2500E+02	6.7838E-07	1.0573E-06	1.0437E-06	1.4821E-06	1.0458E-06	1.2373E-06	1.5626E-06	2.7648E-06
8.3000E+02	6.7243E-07	1.0476E-06	1.0340E-06	1.4683E-06	1.0357E-06	1.2247E-06	1.5466E-06	2.7366E-06
8.3500E+02	6.6657E-07	1.0381E-06	1.0244E-06	1.4546E-06	1.0257E-06	1.2124E-06	1.5309E-06	2.7088E-06
8.7000E+02	6.2769E-07	9.7520E-07	9.6106E-07	1.3645E-06	9.6026E-07	1.1315E-06	1.4276E-06	2.5264E-06
8.7800E+02	6.1948E-07	9.6207E-07	9.4795E-07	1.3458E-06	9.4669E-07	1.1151E-06	1.4065E-06	2.4889E-06
8.8000E+02	6.1753E-07	9.5899E-07	9.4493E-07	1.3414E-06	9.4353E-07	1.1114E-06	1.4017E-06	2.4803E-06
9.3800E+02	5.6482E-07	8.7614E-07	8.6344E-07	1.2249E-06	8.5888E-07	1.0128E-06	1.2726E-06	2.2490E-06
9.4300E+02	5.6061E-07	8.6953E-07	8.5694E-07	1.2156E-06	8.5216E-07	1.0049E-06	1.2624E-06	2.2307E-06
9.6000E+02	5.4666E-07	8.4764E-07	8.3542E-07	1.1848E-06	8.2992E-07	9.7902E-07	1.2286E-06	2.1702E-06
1.0300E+03	4.9533E-07	7.6704E-07	7.5603E-07	1.0716E-06	7.4861E-07	8.8415E-07	1.1055E-06	1.9498E-06
1.1000E+03	4.5219E-07	6.9922E-07	6.8909E-07	9.7642E-07	6.8083E-07	8.0490E-07	1.0033E-06	1.7667E-06
1.2100E+03	3.9560E-07	6.1057E-07	6.0166E-07	8.5218E-07	5.9274E-07	7.0170E-07	8.7105E-07	1.5305E-06
1.2350E+03	3.8433E-07	5.9294E-07	5.8428E-07	8.2750E-07	5.7528E-07	6.8131E-07	8.4494E-07	1.4839E-06
1.5000E+03	2.9069E-07	4.4711E-07	4.4056E-07	6.2363E-07	4.3177E-07	5.1284E-07	6.3133E-07	1.1043E-06
1.5800E+03	2.6944E-07	4.1414E-07	4.0809E-07	5.7760E-07	3.9954E-07	4.7491E-07	5.8363E-07	1.0198E-06

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

** ANNUAL AVERAGE ** **12 MO DATA** TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) 8/4/74 – 8/3/75
DIRECTIONS INDICATED ARE SECTORS FROM WHICH THE WIND IS BLOWING

**CHI/Q FOR RELEASE HEIGHT OF * 0. METERS * (IN SEC PER CU METER) **

Dist, M	NNE	NE	ENE	E	ESE	SE	SSE	S
1.8000E+03	2.2236E-07	3.4128E-07	3.3632E-07	4.7595E-07	3.2859E-07	3.9131E-07	4.7902E-07	8.3514E-07
2.4000E+03	1.4663E-07	2.2488E-07	2.2175E-07	3.1446E-07	2.1922E-07	2.6300E-07	3.2216E-07	5.5790E-07
3.2000E+03	9.5807E-08	1.4699E-07	1.4509E-07	2.0625E-07	1.4558E-07	1.7597E-07	2.1624E-07	3.7215E-07
4.8000E+03	5.2322E-08	8.0820E-08	7.9946E-08	1.1426E-07	8.2365E-08	1.0094E-07	1.2512E-07	2.1472E-07
5.6000E+03	4.1456E-08	6.4311E-08	6.3632E-08	9.1214E-08	6.6351E-08	8.1686E-08	1.0180E-07	1.7501E-07
6.4000E+03	3.3805E-08	5.2791E-08	5.2253E-08	7.5104E-08	5.5113E-08	6.8130E-08	8.5345E-08	1.4692E-07
7.2000E+03	2.8362E-08	4.4334E-08	4.3896E-08	6.3242E-08	4.6768E-08	5.8017E-08	7.3005E-08	1.2583E-07
8.8000E+03	2.0910E-08	3.2894E-08	3.2589E-08	4.7140E-08	3.5316E-08	4.4065E-08	5.5866E-08	9.6492E-08
9.6000E+03	1.8314E-08	2.8892E-08	2.8630E-08	4.1487E-08	3.1255E-08	3.9092E-08	4.9719E-08	8.5955E-08
1.0400E+04	1.6217E-08	2.5660E-08	2.5439E-08	3.6920E-03	2.7955E-08	3.5057E-08	4.4709E-08	7.7358E-08
1.2000E+04	1.3059E-08	2.0786E-08	2.0632E-08	3.0023E-08	2.2935E-08	2.8919E-08	3.7046E-08	6.4194E-08
1.2800E+04	1.1842E-08	1.8901E-08	1.8771E-08	2.7348E-08	2.0975E-08	2.6511E-08	3.4027E-08	5.9003E-08
1.4400E+04	9.9054E-09	1.5889E-08	1.5796E-08	2.3064E-08	1.7815E-08	2.2614E-08	2.9128E-08	5.0570E-08
1.5200E+04	9.1252E-09	1.4672E-08	1.4592E-08	2.1328E-08	1.6527E-08	2.1021E-08	2.7118E-08	4.7107E-08
1.6000E+04	8.4417E-09	1.3603E-08	1.3535E-08	1.9802E-08	1.5391E-08	1.9612E-08	2.5338E-08	4.4039E-08
1.7600E+04	7.3043E-09	1.1820E-08	1.1770E-08	1.7250E-08	1.3483E-08	1.7237E-08	2.2330E-08	3.8850E-08
1.8400E+04	6.8422E-09	1.1091E-08	1.1047E-08	1.6203E-08	1.2696E-08	1.6252E-08	2.1081E-08	3.6691E-08
1.9200E+04	6.4316E-09	1.0441E-08	1.0402E-08	1.5269E-08	1.1993E-08	1.5369E-08	1.9960E-08	3.4752E-08
2.0000E+04	6.0609E-09	9.8536E-09	9.8193E-09	1.4423E-08	1.1355E-08	1.4567E-08	1.8940E-08	3.2987E-08
2.1600E+04	5.4190E-09	8.8345E-09	8.8073E-09	1.2954E-08	1.0243E-08	1.3166E-08	1.7155E-08	2.9897E-08
2.2400E+04	5.1398E-09	8.3902E-09	8.3660E-09	1.2313E-08	9.7558E-09	1.2551E-08	1.6371E-08	2.8538E-08
2.4000E+04	4.6489E-09	7.6079E-09	7.5887E-09	1.1183E-08	8.8942E-09	1.1462E-08	1.4979E-08	2.6127E-08
5.0000E+04	1.6099E-09	2.7114E-09	2.7149E-09	4.0559E-09	3.3665E-09	4.4148E-09	5.8885E-09	1.0330E-08
1.0000E+05	6.1067E-10	1.0546E-09	1.0603E-09	1.6008E-09	1.3721E-09	1.82516-09	2.4669E-09	4.3418E-09

TABLE 2.3-12 (Continued)

Onsite Wind and Stability Summaries with Stability Based on ΔT

** ANNUAL AVERAGE ** **12 MO DATA** TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) 8/4/74 – 8/3/75
DIRECTIONS INDICATED ARE SECTORS FROM WHICH THE WIND IS BLOWING

**CHI/Q FOR RELEASE HEIGHT OF * 0. METERS * (IN SEC PER CU METER) **

Dist, M	SSW	SW	WSW	W	WNW	NW	NNW	N
6.0500E+02	5.2451E-06	5.0812E-06	3.2637E-06	2.6124E-06	1.0623E-06	1.3064E-06	1.0222E-06	9.0741E-07
6.2000E+02	5.0346E-06	4.8812E-06	3.1389E-06	2.5128E-06	1.0224E-06	1.2589E-06	9.8533E-07	8.7588E-07
6.3500E+02	4.8371E-06	4.6935E-06	3.0215E-06	2.4191E-06	9.8481E-07	1.2140E-06	9.5058E-07	8.4608E-07
6.4000E+02	4.7740E-06	4.6335E-06	2.9340E-06	2.3892E-06	9.7279E-07	1.1996E-06	9.3944E-07	8.3651E-07
6.7000E+02	4.4216E-06	4.2977E-06	2.7736E-06	2.2212E-06	9.0534E-07	1.1188E-06	8.7677E-07	7.8261E-07
6.8000E+02	4.3132E-06	4.1943E-06	2.7087E-06	2.1694E-06	8.8451E-07	1.0938E-06	8.5737E-07	7.6587E-07
7.2000E+02	3.9193E-06	3.8180E-06	2.4720E-06	1.9804E-06	8.0839E-07	1.0022E-06	7.8624E-07	7.0436E-07
7.2500E+02	3.8741E-06	3.7748E-06	2.4447E-06	1.9586E-06	7.9962E-07	9.9163E-07	7.7802E-07	6.9724E-07
7.3000E+02	3.8297E-06	3.7323E-06	2.4180E-06	1.9373E-06	7.9099E-07	9.8122E-07	7.6993E-07	6.9022E-07
7.4000E+02	3.7433E-06	3.6496E-06	2.3658E-06	1.8956E-06	7.7418E-07	9.6093E-07	7.5416E-07	6.7653E-07
8.1500E+02	3.1834E-06	3.1123E-06	2.0257E-06	1.6239E-06	6.6438E-07	8.2795E-07	6.5065E-07	5.8631E-07
8.2500E+02	3.1188E-06	3.0003E-06	1.9863E-06	1.5924E-06	6.5163E-07	8.1244E-07	6.3857E-07	5.7574E-07
8.3000E+02	3.0873E-06	3.0200E-06	1.9670E-06	1.5770E-06	6.4539E-07	8.0486E-07	6.3266E-07	5.7056E-07
8.3500E+02	3.0563E-06	2.9901E-06	1.9480E-06	1.5618E-06	6.3925E-07	7.9740E-07	6.2684E-07	5.6546E-07
8.7000E+02	2.8524E-06	2.7937E-06	1.8230E-06	1.4619E-06	5.9875E-07	7.4806E-07	5.8836E-07	5.3170E-07
8.7800E+02	2.8106E-06	2.7532E-06	1.7970E-06	1.4411E-06	5.9045E-06	7.3786E-07	5.8041E-07	5.2463E-07
8.8000E+02	2.8018E-06	2.7439E-06	1.7910E-06	1.4362E-06	5.8857E-07	7.3550E-07	5.7858E-07	5.2296E-07
9.3800E+02	2.5442E-06	2.4931E-06	1.6279E-06	1.3053E-06	5.3785E-07	6.7216E-07	5.2929E-07	4.7816E-07
9.4300E+02	2.5239E-06	2.4732E-06	1.6149E-06	1.2951E-06	5.3380E-07	6.6710E-07	5.2535E-07	4.7458E-07
9.6000E+02	2.4566E-06	2.4075E-06	1.5721E-06	1.2608E-06	5.2041E-07	6.5037E-07	5.1231E-07	4.6273E-07
1.0300E+03	2.2110E-06	2.1675E-06	1.4156E-06	1.1357E-06	4.7117E-07	5.8881E-07	4.6431E-07	4.1913E-07
1.1000E+03	2.0066E-06	1.9876E-06	1.2850E-06	1.0318E-06	4.2979E-07	5.3708E-07	4.2391E-07	3.8249E-07
1.2100E+03	1.7422E-06	1.7090E-06	1.1158E-06	8.9697E-07	3.7568E-07	4.6941E-07	3.7096E-07	3.3448E-07
1.2350E+03	1.6899E-06	1.6579E-06	1.0823E-06	8.7030E-07	3.6491E-07	4.5594E-07	3.6040E-07	3.2492E-07
1.5000E+03	1.2626E-06	1.2392E-06	8.0839E-07	6.5150E-07	2.7577E-07	3.4443E-07	2.7281E-07	2.4563E-07
1.5800E+03	1.1671E-06	1.1456E-06	7.4715E-07	6.0249E-07	2.5560E-07	3.1919E-07	2.5293E-07	2.2766E-07

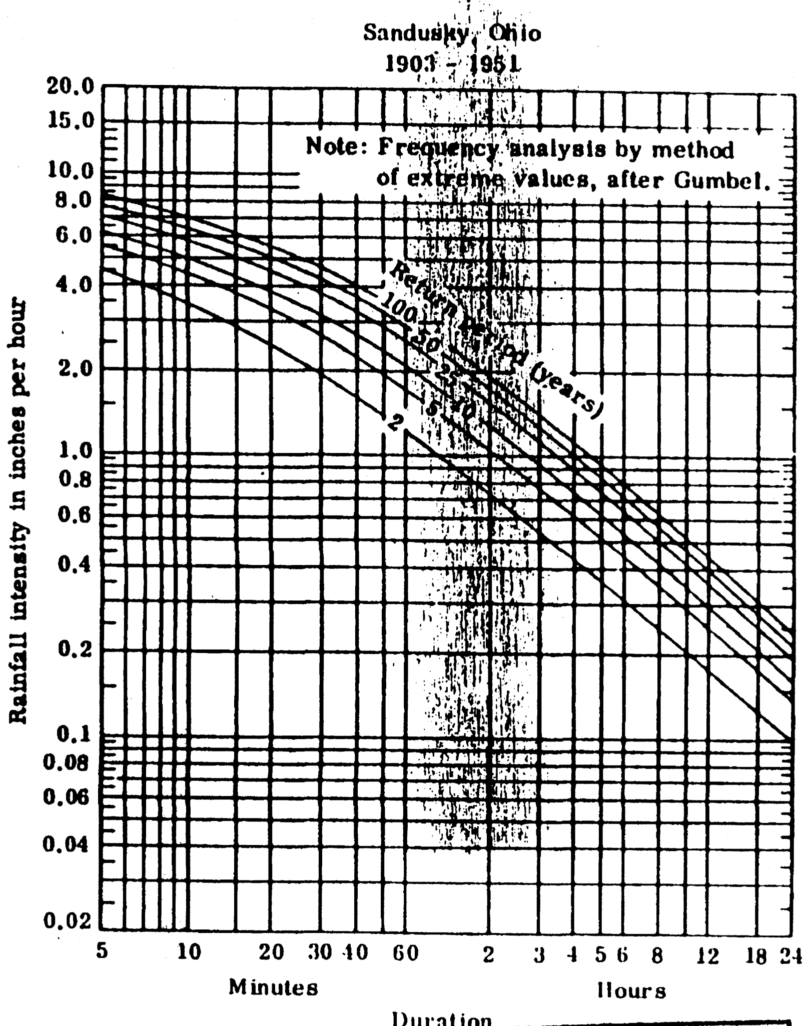
TABLE 2.3-12 (Continued)

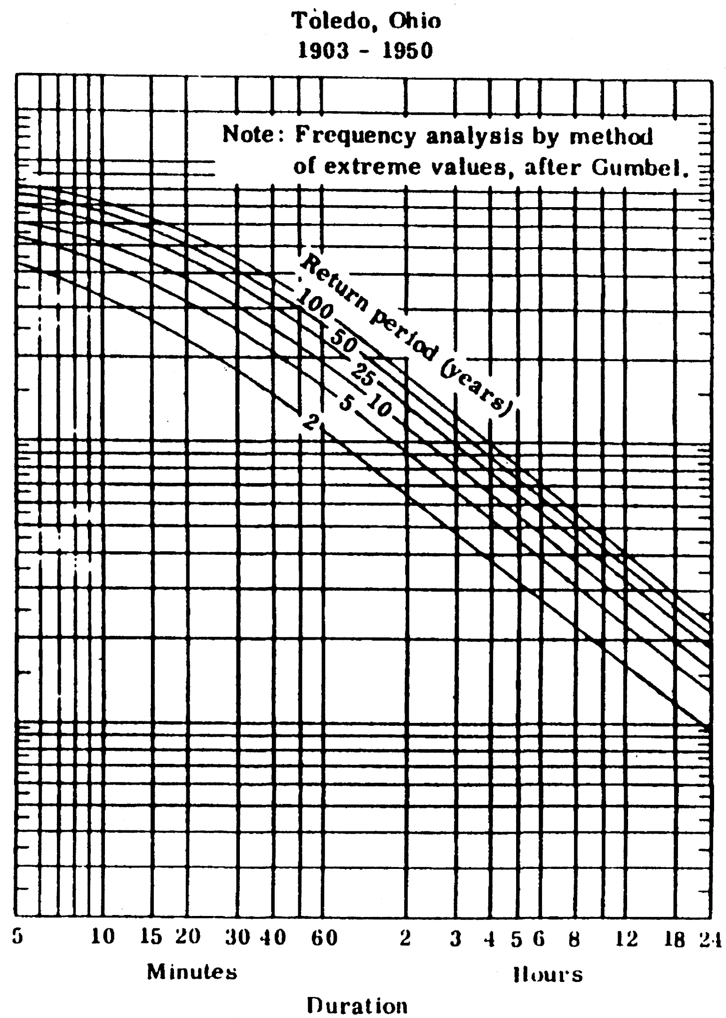
Onsite Wind and Stability Summaries with Stability Based on ΔT

** ANNUAL AVERAGE ** **12 MO DATA** TOLEDO - EDISON 35 FT WINDS ($\Delta T_{250'-35'}$) 8/4/74 – 8/3/75
DIRECTIONS INDICATED ARE SECTORS FROM WHICH THE WIND IS BLOWING

**CHI/Q FOR RELEASE HEIGHT OF * 0. METERS * (IN SEC PER CU METER) **

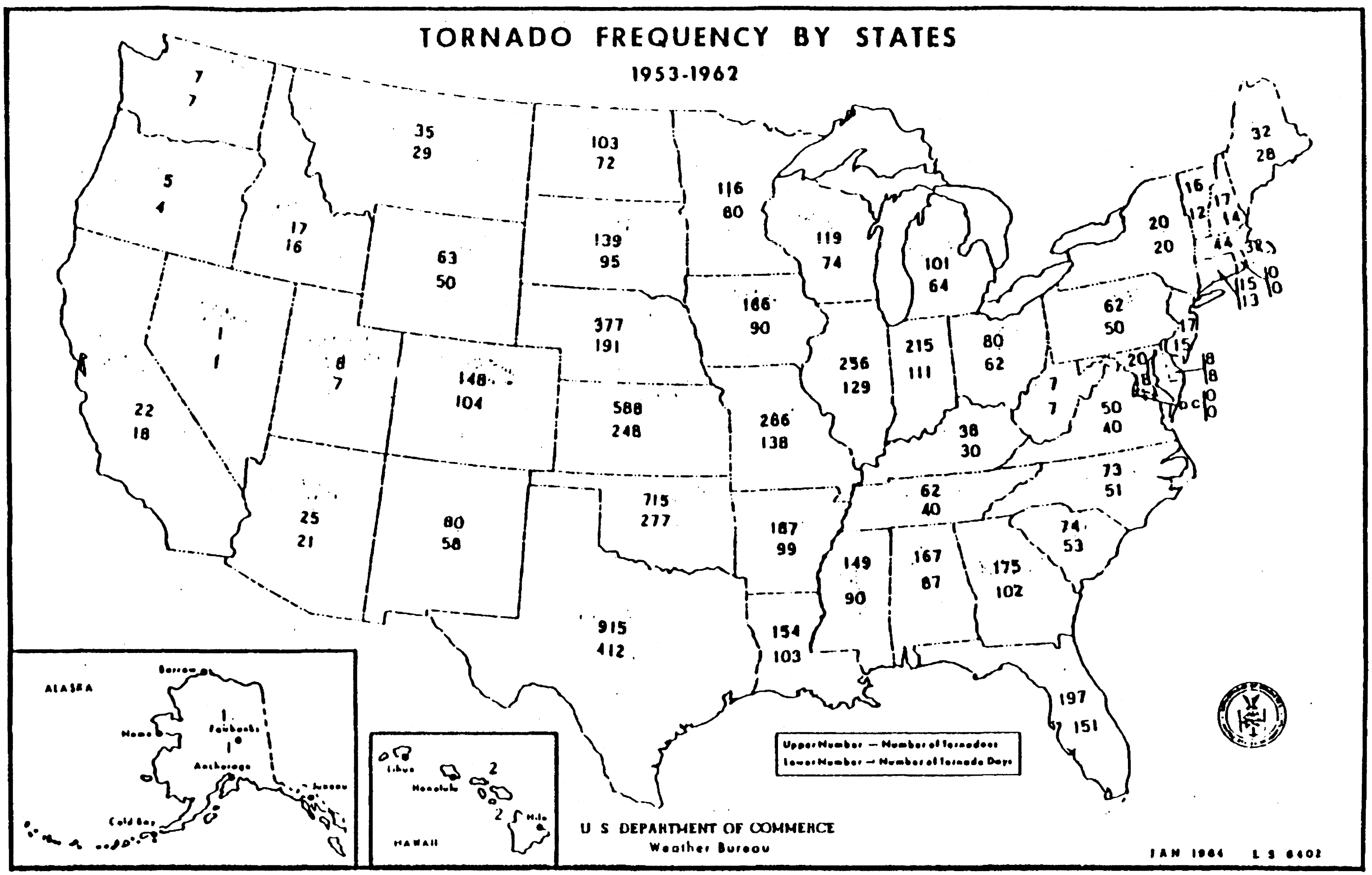
Dist, M	SSW	SW	WSW	W	WNW	NW	NNW	N
1.8000E+03	9.5776E-07	9.4016E-07	6.1279E-07	4.9480E-07	2.1095E-07	2.6334E-07	2.0889E-07	1.8787E-07
2.4000E+03	6.4047E-07	6.2797E-07	4.0592E-07	3.3100E-07	1.4055E-07	1.7472E-07	1.3852E-07	1.2419E-07
3.2000E+03	4.2719E-07	4.1825E-07	2.6783E-07	2.2071E-07	9.2997E-08	1.1507E-07	9.1079E-08	8.1416E-08
4.8000E+03	2.4513E-07	2.3875E-07	1.5113E-07	1.2538E-07	5.2098E-08	6.3867E-08	5.0465E-08	4.4765E-08
5.6000E+03	1.9894E-07	1.9323E-07	1.2200E-07	1.0111E-07	4.1697E-08	5.0926E-08	4.0226E-08	3.5564E-08
6.4000E+03	1.6638E-07	1.6122E-07	1.0151E-07	8.4096E-08	3.4423E-08	4.1896E-08	3.3074E-08	2.9153E-08
7.2000E+03	1.4203E-07	1.3734E-07	8.6277E-08	7.1448E-08	2.9053E-08	3.5251E-08	2.7814E-08	2.4451E-08
8.8000E+03	1.0832E-07	1.0440E-07	6.5334E-08	5.4062E-08	2.1738E-08	2.6239E-08	2.0683E-08	1.8099E-08
9.6000E+03	9.6269E-08	9.2652E-08	5.7892E-08	4.7887E-08	1.9161E-08	2.3077E-08	1.8183E-08	1.5880E-08
1.0400E+04	8.6460E-08	8.3095E-08	5.1841E-08	4.2859E-08	1.7080E-08	2.0524E-08	1.6166E-08	1.4088E-08
1.2000E+04	7.1486E-08	6.8518E-08	4.2628E-08	3.5193E-08	1.3934E-08	1.6671E-08	1.3123E-08	1.1385E-08
1.2800E+04	6.5599E-08	6.2800E-08	3.9023E-08	3.2196E-08	1.2711E-08	1.5178E-08	1.1945E-08	1.0341E-08
1.4400E+04	5.6061E-08	5.3555E-08	3.3207E-08	2.7364E-08	1.0747E-08	1.2787E-08	1.0059E-08	8.6761E-09
1.5200E+04	5.2155E-08	4.9775E-08	3.0834E-08	2.5393E-08	9.9497E-09	1.1819E-08	9.2954E-09	8.0039E-09
1.6000E+04	4.8698E-08	4.6434E-08	2.8739E-08	2.3654E-08	9.2477E-09	1.0968E-08	8.6246E-09	7.4143E-09
1.7600E+04	4.2864E-08	4.0803E-08	2.5214E-08	2.0731E-08	8.0717E-09	9.5463E-09	7.5038E-09	6.4312E-09
1.8400E+04	4.0444E-08	3.8475E-08	2.3758E-08	1.9528E-08	7.5882E-09	8.9640E-09	7.0446E-09	6.0305E-09
1.9200E+04	3.8272E-08	3.6388E-08	2.2455E-08	1.8453E-08	7.1562E-09	8.4447E-09	6.6351E-09	5.6740E-09
2.0000E+04	3.6298E-08	3.4492E-08	2.1271E-08	1.7477E-08	6.7650E-09	7.9748E-09	6.2647E-09	5.3518E-09
2.1600E+04	3.2847E-08	3.1181E-08	1.9206E-08	1.5775E-08	6.0845E-09	7.1589E-09	5.6215E-09	4.7932E-09
2.2400E+04	3.1332E-08	2.9728E-08	1.8301E-08	1.5029E-08	5.7871E-09	6.8028E-09	5.3410E-09	4.5499E-09
2.4000E+04	2.8645E-08	2.7155E-08	1.6700E-08	1.3709E-08	5.2623E-09	6.1753E-09	4.8466E-09	4.1217E-09
5.0000E+04	1.1166E-08	1.0489E-08	6.3812E-09	5.2221E-09	1.9344E-09	2.2278E-09	1.7411E-09	1.4528E-09
1.0000E+05	4.6494E-09	4.3387E-09	2.6171E-09	2.1376E-09	7.7246E-10	8.7599E-10	6.8232E-10	5.5995E-10





DAVIS-BESSE NUCLEAR POWER STATION
RAINFALL INTENSITY, DURATION AND FREQUENCY CURVES
FIGURE 2.3-1

REVISION 0



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DAVIS-BESSE NUCLEAR POWER STATION
TORNADO FREQUENCY BY STATES
FIGURE 2.3-2

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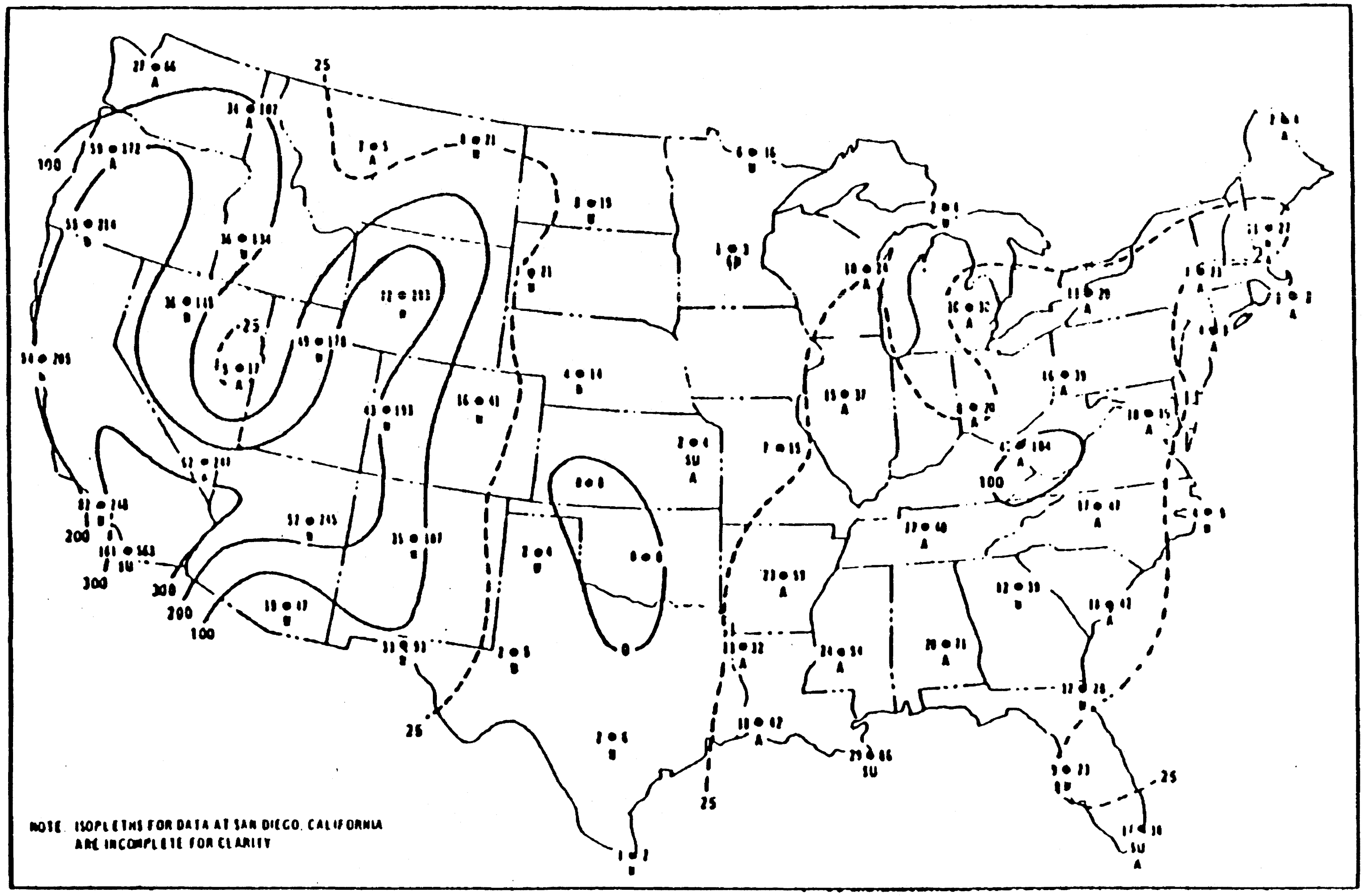
JULY 1982



Isopleths of total number of episode-days in 5 years with mixing heights $\leq$ 1500 m, wind speeds $\leq$ 4.0 m sec^{-1}, and no significant precipitation - - - - - - - - - for episodes lasting at least 2 days. Numerals on left and right give total number of episodes and episode-days, respectively. Season with greatest number of episode-days indicated as W (winter), SP (spring), SU (summer), or A (autumn).

DAVIS-BESSE NUCLEAR POWER STATION
ISOPLETH DATA
FIGURE 2.3-3

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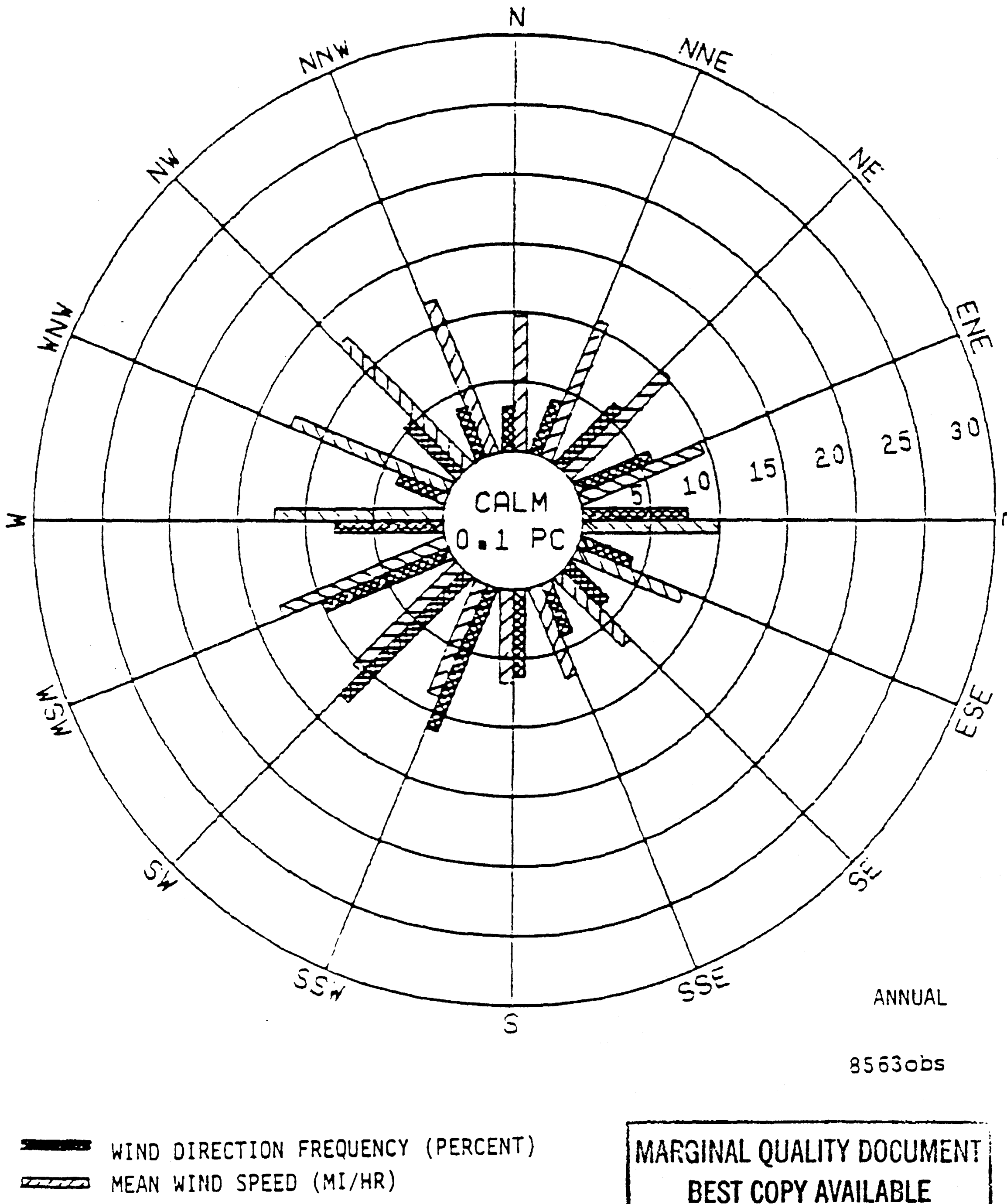
JULY 1982



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DAVIS-BESSE NUCLEAR POWER STATION
DAVIS-BESSE SITE ANNUAL WIND DISTRIBUTIONS
AT THE 35-FT LEVEL, AUGUST 4, 1974-AUGUST 3, 1975
FIGURE 2.3-4

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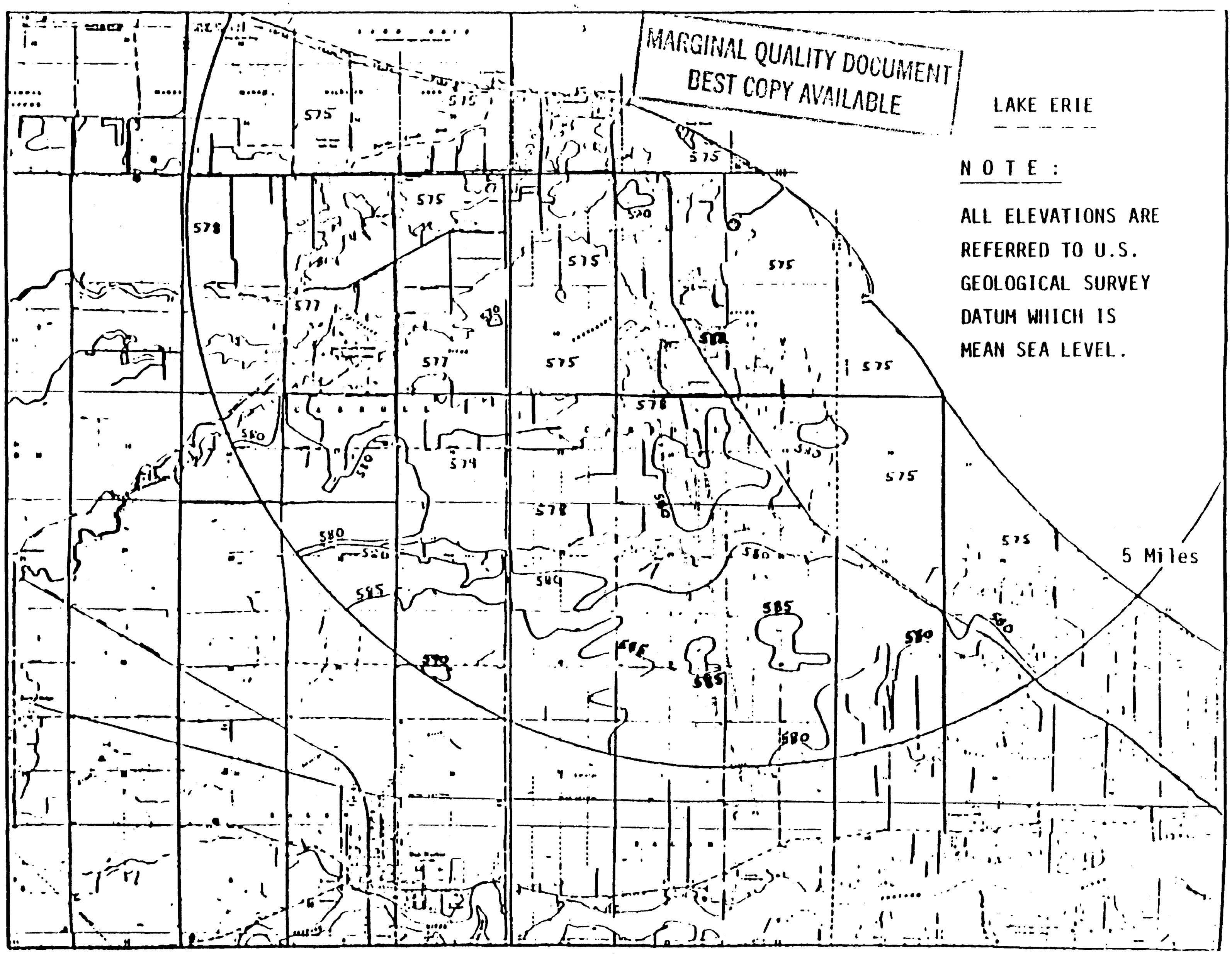
JULY 1982



DAVIS-BESSE NUCLEAR POWER STATION
ELEVATIONS OF SITE AND
SURROUNDING AREA
FIGURE 2.3-5

REVISION 0
JULY 1982

2.4 HYDROLOGY

2.4.1 Hydrologic Description

2.4.1.1 Site and Facilities

The general topography of the original site was virtually featureless with no natural promontories. The major hydrological features of the terrain are the broad expanse of Lake Erie to the north and east and the Toussaint River which flows east into the lake along the south side of the site.

All elevations herein are referenced to the International Great Lakes Datum (I.G.L.D.) which was established in 1955 by the U.S. Dept. of Commerce. It is a measure, in feet, above the mean water level in the Gulf of St. Lawrence at Father Point, Quebec. Using this reference point, the low water datum set for Lake Erie is 568.6 feet I.G.L.D.

The original topography of the site was relatively flat with elevations varying from the lake elevation (low water datum of 568.6 feet I.G.L.D.) to approximately six feet above the lake level. The site had very little slope and there was and there still remains approximately 733 acres of marsh in its eastern and northern portions near the lake. A narrow beach ridge, elevation approximately 575 feet (I.G.L.D.), separates this marsh area from the lake. The remaining 221 acres of the southwest portion of the site originally was three to six feet above the lake level. The station structures are located on the eastern edge of this upland section. The original topography of the site is shown on Figure 2.4-1.

The northern portion of the upland section of the site originally drained into the marsh along the north portion of the site. This marsh then drained to Lake Erie through the canal that entered the lake between Long Beach and Sand Beach. The southern portion of the upland section of the site originally drained into the Toussaint River through a ditch that flows along the site's south boundary. The marsh area in the eastern section of the site also drained through this ditch into the Toussaint River. The two marsh areas were separated by the road that extended from the upland section out to the beach ridge along the lake. However, the water levels in the northern and eastern marsh areas fluctuated with the lake level, as they were directly connected to the lake and the Toussaint River through canals and ditches. See Section 2.4.1.2 for a description of the Toussaint River water levels.

The construction of the nuclear station has changed the topography and drainage of the site as follows:

a. The area occupied by the station structures has been built up to a grade elevation of 584 feet (I.G.L.D.). The station structures are protected against high water levels up to an elevation of 585 feet (I.G.L.D.). In addition, a wave protection dike has been installed along the north, east, and a small portion along the south sides of this built up area to an elevation of 591 feet (I.G.L.D.). The grade elevation around the 345kV switchyard and immediately adjacent to the cooling tower has been built up to 582 feet (I.G.L.D.). All surface water from these elevated areas will be collected and carried in storm drains to the ditch that empties into the Toussaint River or to the ditches that empty into the adjacent marsh areas.

b. An intake canal has been installed between the intake structure at the nuclear station and the beach ridge. This intake canal is connected to Lake Erie with an eight foot diameter underground and underwater pipe that extends 3,300 feet out

into the lake. The top of the dikes on either side and the lake end of this canal in the marsh area are built up to an elevation of 579 feet (I.G.L.D.) and at the intake structure these dikes are built up to 591 feet (I.G.L.D.).

c. A new marsh dike has been installed along the north site boundary to an elevation of 575 feet (I.G.L.D.). Also the existing dikes around the marsh area in the southeastern section of the site have been repaired and raised to an elevation which varies between 573 and 576 feet (I.G.L.D.). These dikes isolate the marsh areas of the site from Lake Erie and were installed at the request of the U.S. Fish and Wildlife Service to control the marsh water levels. Pumps have been installed to control this water level.

d. Three borrow pits and one stone quarry have been dug to provide fill material. These pits and quarry occupy about 46 acres of the upland section of the site and are now filled with water to a level corresponding to approximate lake elevation.

A topographic map showing all changes to the site topography and drainage is shown on Figure 2.4-2.

2.4.1.1.1 Site Drainage Facilities

There are two main drainage facilities in the station drainage system:

a. Roof drain system

b. Storm sewer system

The roof drain system is designed to be completely isolated from any floor or equipment drain line. The roof rain runoff is collected by roof drains and flows by gravity through down spouts. There is no interruption or connection to these down spouts.

All of the main power block structures' down spouts penetrate the ground, then are connected to the station storm sewer system. The main power block roof drains are so located and spaced that, if for any possible reason one or two of the drains are stopped, the other roof drains nearby will be able to adequately drain the rainfall runoff. However, the selected type of roof drains is such that the possibility of any stopped-up drainage is very remote. In addition to the roof drains, there are overflow pipes located in roof parapets. The purpose of these pipes is primarily for overflow of any built-up water due to stoppage in roof drains or rainfall intensities higher than design intensity. All plumbing system pipes in Class I structures are supported by Seismic Class I hangers. The station storm sewer system collects all site ground rain runoff effluent in addition to the roof drainage. There are many sewer manholes, catch basins, collection boxes, and road drains to collect the maximum possible rainfall runoff. All these facilities are connected by underground sewer piping. The final manhole will discharge the runoff water to the existing ditch, which eventually empties into the Toussaint River.

There are cleanup boxes and provisions at each line to maintain the system and ensure that it operates properly. In the isolated areas, the clean rainfall runoff water is collected in the catch basins and emptied to the marsh area. As there is no regular flow associated with these systems, provisions have been made, where necessary, to prevent damage to pipes and pumps during freezing weather. Electric immersion heaters were installed in the settling basin pump pit and overflow pit in order to maintain the water above freezing when there is no flow.

2.4.1.2 Hydrosphere

2.4.1.2.1 Lake Hydrology

Lake Erie is one of the smallest of the Great Lakes. It is, however, 241 miles long from west to east and has an average width of 57 miles. It has a maximum depth of 210 feet, an average depth of 60 feet and covers an area of 9,910 square miles. About 2,200 square miles of this lake are within a 50 mile radius of the station site and the average depth within this radius is about 25 feet. The total volume of water in Lake Erie is approximately 110 cubic miles and the annual flow of water out of Lake Erie is about 40% of this total volume. The major hydrologic features of the surrounding area are shown on Figure 2.1-1.

2.4.1.2.2 Characteristics of Streams

The Toussaint River empties into Lake Erie southeast of the station's site. This stream flows close to the south of the site and becomes Toussaint Creek about six miles upstream from its mouth. No water is taken from this stream for use in this station and only station storm sewer water and turbine building drains can be discharged into it.

The headwaters of the Toussaint Creek have a maximum elevation of about 670 feet (I.G.L.D.). This stream has a drainage area of about 143 square miles and an average slope of about two feet per mile. There are no dams on this stream. The lower six miles of the stream are much wider than the remainder except for a narrow mouth where it empties into Lake Erie and, as a result, its level in this wider section is controlled by the level of Lake Erie.

2.4.1.2.3 Groundwater

The site is underlain by a glaciolacustrine deposit and a till deposit which overlie sedimentary bedrock. The soil deposits, which essentially consist of silty clay have very low permeability and are considered impervious. Their combined thickness is on the order of 20 feet. The bedrock consists of the Tymochtee formation underlain by the Greenfield formation. These formations consist of nearly horizontal beds of argillaceous dolomite with shale, gypsum, and anhydrite, to a depth of at least 200 feet below ground surface. The presence of the impervious soil deposits has produced an artesian groundwater condition in the bedrock, which is the aquifer in the site locality.

See Subsection 2.4.13 for further details.

2.4.1.2.4 Surface Water Users

The following information pertains to the use of groundwater and was provided as part of the original safety analysis. Current conditions are not foreseen to differ significantly, however it is not practical to continuously update this section to reflect the current status. Therefore, this section is considered historical.

The primary source of potable water in the area is Lake Erie. The nearest potable water supplies are as follows:

Owner	Distance from Station Discharge to Intake (miles)	Gallons/Day Usage ($x10^6$)
Erie Industrial Park	3.6	0.2
Camp Perry	4.6	0.1975
Port Clinton	8.6	1.5
Oregon	12.0	4.5
Toledo	12.0	77.8
Carroll Township Water	3.0	0.6

The relative location of these potable water intakes with respect to the station discharge point is shown in Figure 2.4-3.

The Sand Beach community is located along the beach ridge commencing at the northern site boundary at the shoreline. At the time of original construction this community contained 122 residences, principally summer. Approximately 50 percent of them obtained household water for all purposes from beach wells located in the lake front sand. Another approximately 25 percent used this type well for all but potable water use. Twenty-one of the twenty-four permanent residences used beach wells for all- purpose use.

2.4.2 Floods

The station structures are over 3,000 feet from the Lake Erie shoreline. The Emergency Feedwater Facility grade floor is at elevation 586' (I.G.L.D.). All other station grade floors are at elevation 585 feet (I.G.L.D.) and the station is designed for normal power generation at water levels from 562 feet (I.G.L.D.) up to this elevation. In addition, a breakwater dike is installed along the north and east side of the station to protect it against flooding due to waves and wave run-up during a probable maximum meteorological event.

2.4.2.1 Flood History

As described in Subsection 2.4.1.1, the original topography of the site is relatively flat and rises only about six feet above the mean low lake level. As a result, it is expected that the site areas that have not been raised above the original grade may be flooded by high water levels in Lake Erie. The highest recorded static water level (no wind tide or wave effect) on Lake Erie recorded by the U.S. Department of Commerce, National Ocean Survey was 573.51 feet (I.G.L.D.) in June 1973. The highest water level (including wind tide effect but no wave or ice jam effects) recorded in Maumee Bay near Toledo by the U.S. Army Corp of Engineers was 576.67 feet (I.G.L.D.) on April 9, 1973.

The Toussaint River empties into Lake Erie about 1-1/4 miles to the southeast of the station. As described in Subsection 2.4.1.2.2 the level of the lower six miles of this stream is controlled by the level of Lake Erie. As a result, no record of flooding on the station's site due to high water on this stream alone has been recorded.

There are no other streams, lakes, or reservoirs that have caused flooding at the site.

2.4.2.1.1 Analysis of November, 1972 Storm

On November 13, 1972, a storm moved into the Lake Erie drainage basin. This storm was classified as being of moderate intensity by the National Weather Service (Snider oral communication). However, the total effect of the storm was greatly increased by the existing high water level of Lake Erie. In November the lake was 3.5 feet above the Low Water Datum of 568.6 feet (I.G.L.D.). The combination of the higher lake level and moderate winds initiated wide-spread flooding, and subsequent local evacuation within the low-lying areas along the western and southwestern shore after the storm winds shifted from south to northeast.

The maximum wind setup on Lake Erie occurred at Toledo, Ohio where the stillwater level rose to very nearly 576.0 feet (I.G.L.D.), or a total of 7.4 feet above Low Water Datum. This wind setup was driven by northeast winds with a directional duration of approximately 12 hours. The maximum wind speed recorded at Toledo airport was 25 knots. This wind speed, when corrected for distance overland, is in agreement with the wind speed recorded at the Toledo Coast Guard Station as given in Reference 42.

For most of November 12, winds were light (4 knots) and out of the southwest. Very late on the 12th and throughout the 13th, winds shifted gradually to northwest winds then to northeast. By midday on November 13, the northeast winds were established and velocity increased to 20 knots. Water level began rising at Toledo at 0400 hours on November 13.

The maximum wind speed occurred early November 14th. At noon on November 14, the wind direction began changing to North, and by 1900 hours, the still water level at Toledo reached its maximum of 576.00 feet (I.G.L.D.). The wind direction remained northerly throughout the 15th of November while speed varied from 5 to 14 knots. Secondary and tertiary seiches were experienced on the 15th, but they decayed quite rapidly. By November 16, 1972, the water level stabilized at approximately 572.4 feet (I.G.L.D.).

Moving away from Toledo, along both the Michigan and Ohio shorelines, water level gauges recorded proportionally lower maximum wind set up. At the Fermi power plant north of Monroe, Michigan, the maximum set up was recorded at elevation 574.9 feet (I.G.L.D.), while set up elevations of 575.12 (I.G.L.D.) and 574.3 (I.G.L.D.) were measured at Marblehead and Cleveland, Ohio, respectively.

The generation of waves is a function of the speed, the fetch, and the duration of the wind and the depth of water. The wind speeds used to calculate the wave height and period generated during this storm were measured at the Toledo Express Airport. These wind speeds were increased by a factor of 1.43 to obtain overwater velocities. The fetches were aligned with the wind directions and reduced to effective fetch lengths in accordance with procedures in Reference 25. The depths were obtained by superposition of the measured high water levels on the low water depths given on Lake Survey Chart No. 39. Several critical combinations of these parameters were analyzed for this storm and are presented in Table 2.4-1. Significant wave heights between 3.0 feet and 5.6 feet and maximum wave heights between 5.6 feet and 10.5 feet high could have been generated during this storm. These wave heights hindcasted for the western basin of the lake are consistent with the 12 foot maximum waves reported for the eastern basin in Reference 25.

However, the waves which could affect the Davis-Besse site would be limited in height by the available depth of water over the gradually sloping beach at the site. The toe of the beach berm is at approximately El. 570 feet (I.G.L.D.). Waves larger than those supportable at this point will break seaward of the shoreline and can be neglected.

From Table 2.4-1, it can be seen that the largest supportable wave at the toe of the beach berm occurs during the maximum stillwater level of 576.0 feet (I.G.L.D.). Conservatively, assuming that these two conditions were coincident in time, the significant wave is 5.6 feet high with a significant period of 3.9 seconds and the maximum generated wave is 10.5 feet high. These waves would break seaward of the beach berm. The maximum supportable wave is 4.7 feet high and could have a maximum period of 4.9 seconds. It would produce a wave run-up of 3.7 feet on the beach berm. The maximum run-up elevation which could have been reached during this storm is 579.7 feet (I.G.L.D.). This elevation is considerably less than the station grade and roadway elevations at the Davis-Besse site of 584.0 feet (I.G.L.D.) and the probable maximum meteorological event (PMME) water levels (Subsection 2.4.5.1).

2.4.2.1.2 Analysis of April, 1973 Storm

Another storm moved into the Lake Erie basin on April 19, 1973. Although this storm was less intense than the November 1972 storm, its total effect was nearly equal to the November storm because of the extremely high static lake levels. In April, the mean lake level at Toledo was measured by the U.S. Lake Survey at +4.76 feet above the Low Water Datum of 568.6 feet (I.G.L.D.).

The maximum wind set up associated with this spring storm was measured as +3.31 feet at Toledo which brought the total stillwater level to 576.67 feet (I.G.L.D.). This is 0.69 feet higher than the level reached by the November 1972 storm.

The wind velocities measured at the Toledo Express Airport ranged from 4 to 17 knots during the storm. The wind speeds measured at Davis-Besse are in agreement with these velocities after applying a correction for overland effect. During this storm, the highest wind velocities were directed offshore and therefore would not have directed the maximum wave action at the Davis-Besse site.

Calculations indicate that significant waves as high as 3.1 feet with a period of 3.1 seconds and maximum waves as high as 5.8 feet could have been generated (Table 2.4-1). As in the case of the November 1972 storm, the wave action at the site is governed by the maximum supportable wave at the toe of the beach berm during the peak stillwater level of 576.7 feet (I.G.L.D.). The maximum supportable wave is 5.2 feet high and could have a maximum period of 3.9 seconds. It would produce a wave run-up of 3.2 feet. The maximum run-up elevation which could have been reached during this storm is 579.9 feet (I.G.L.D.). This elevation is considerably less than the station grade and roadway elevations at the Davis-Besse site of 584.0 feet (I.G.L.D.).

2.4.2.2 Flood Design Consideration

2.4.2.2.1 Lake Flooding

The static water levels in the western basin of Lake Erie are affected by long term and annual cyclic variations in the mean monthly level from the mean low water level, and short period variations in the daily level from the monthly mean level due to wind tides and seiches. The subject of lake levels has been investigated and the results of this investigation are summarized in this section.

Water level records for Lake Erie have been gathered since 1860. Various planes of reference have been used during the period of record, and each of these reference planes has a correction factor which must be applied when converting to the I.G.L.D. water levels. The U.S.

Army Corps of Engineers, Lake Survey Center, publishes a Monthly Bulletin of Lake Levels which illustrates the monthly record high and low levels for each of the Great Lakes with respect to the I.G.L.D. No conversion factor must be utilized when referring to these monthly bulletins.

The water level gauges positioned around Lake Erie do not have the capability of recording wave heights. It is necessary to compute the maximum waves possible using available physical data. When considering protection of the power plant against wave action, the wave hindcasting techniques employed are those prescribed in Reference 25.

Prior to May and June of 1973, the maximum variation in the mean monthly level of Lake Erie was 4.2 feet above datum and 1.2 feet below datum. During June 1973, the all-time high lake level was recorded as 4.9 feet above datum while the lower limit remained unchanged. At the Davis-Besse site, a probable maximum variation of 4.8 feet above and 1.5 feet below datum has been used. The high water mark for 1974 was reached during May and June, after the early summer rains. The level of Lake Erie met, but did not exceed, the record level reached in 1973.

The short period variations in the daily level from the monthly mean level are due to both a lengthwise wind tide which produces the greatest disturbance of water level and a transverse seiche in the west end of Lake Erie which can oscillate between the northern and southern shores. Hunt in Reference 40 reported that a transverse seiche of 0.8 foot has been recorded in the western basin of Lake Erie, and we are using a probable maximum value of 1.0 foot at Davis-Besse. A uninodal lengthwise seiche on Lake Erie can contribute some increment to the wind tide, but the maximum amplitude of the uninodal seiche cannot coincide with the maximum wind tide.

A probable maximum meteorological event was used to determine the maximum rise in lake level due to wind tides. This meteorological event would have a maximum ENE wind at any one location of 100 miles per hour for a 10-minute period, and the wind speed could exceed 70 miles per hour during the six hour period both before and after the maximum wind speed. The procedure developed by Platzman (ref. 3) was applied to this storm to determine the maximum wind tide rise at Toledo. A report on a verification study on this Platzman procedure is given in Appendix 2D.

Since Davis-Besse site is located about 80% of the way from the wind tide node (point in the lake where no wind tide change in lake level occurs) to Toledo, wind tide variations at Davis-Besse were reduced by 20% from Toledo wind tides. This procedure gave a maximum wind tide rise with ENE winds of 9.3 feet.

For the probable maximum high water level condition at the site the 9.3 foot wind tide could occur at the time of the 4.8 foot long term high monthly mean lake level, and under conditions where the transverse seiche would be adding one foot of lake elevation. The total would be 15.1 feet above the low water datum or a probable maximum high static water level of 583.7 feet (I.G.L.D.).

Wind-generated waves are limited in their dimensions by wind velocity, by fetch (open-water distance available for wind action), by water depth, and by duration of the wind. Higher wind velocities, longer fetches, deeper water, and longer wind durations all increase the heights, lengths, and velocities of the waves. Neither wind velocity nor duration of wind are subject to control by the lake basin, but fetch and depth are a physical characteristic of the lake basin. At Davis-Besse the available fetch plays an important part in the height of the maximum wave that might arrive at the station, on top of the maximum static high water level from other causes.

The probable maximum high water that could occur at Davis-Besse is predominantly the result of wind tide under prolonged strong wind from the ENE. The station's site is in the western basin of Lake Erie, and wind-waves generated by ENE winds over the rest of the lake find their access to the western basin from the central basin. These parts of waves from the eastern parts of the lake that succeed in passing through the islands are damped, refracted, and reflected into a confused sea around the western sides of the islands. From here the NNE wind must construct the maximum wave that will bear upon Davis-Besse. Toward NNE from the station's site, the effective fetch is 23 nautical miles. See Subsection 2.4.5.2 and Figure 2.4-4 for further details.

Using the equations and curves developed in Reference 25, the ENE 100-miles-per-hour winds associated with the probable maximum meteorological event would produce a maximum wave height (difference between wave crest and trough) of 10.7 feet at the lake's normal shoreline. Other equations and curves in Reference 25 indicate that these waves would break in 115 feet of water. These larger waves generating in the lake will break when they meet the normal shoreline, as the ground rises to elevation of above 575 feet (I.G.L.D.) and higher along the shoreline. However, smaller waves generated in the lake up to a height of 6.0 feet would pass over the beach without breaking at the maximum probable static water level. These smaller waves will build up to a maximum height of about 8.5 feet in the marsh area and will break when they reach the elevated area around the station. Figure 2.4-5 shows two vertical sections of the area between the station and the lake shore. The finished grade and roadways around the station are built up to about elevation 584 feet (I.G.L.D.) for a distance of 250 feet to the east and north of the building. This elevated area around the station is protected along the north, east and a small portion along the south sides by an earthfill breakwall built up to 591.0 feet (I.G.L.D.) to protect against the wave and wave run-up. This breakwall is about 15 feet wide at the top. The lakeward side of the breakwall and the banks of the built-up area have a three to one slope and is protected against wave action with riprap. The maximum wave run-up on this break wall will be 6.6 feet above the probable maximum static water level of 583.7 feet (I.G.L.D.). This will give a maximum water run-up level on the breakwall of 590.3 feet (I.G.L.D.). As a result, no large unbroken waves will reach the station's buildings and none will overtop the wave protection dike.

The station's ground floor elevation of 585 feet (I.G.L.D.) will protect the station against the maximum probable static water level of 583.7 feet (I.G.L.D.). Its location, about 3,000 feet from the shoreline, the elevated land along the shoreline, and the breakwater at the station will protect the station against wave action at the maximum probable water level.

The intake structure is designed to accept the wave action directly. The cooling tower is located outside the diked area, and could be subjected to wave action which might require it to be taken out of service. In this event, the station can be brought to a safe and orderly shutdown condition and maintained in this condition since all other systems are fully protected. For basic design criteria of the intake structure and other Class I structures in regard to the maximum possible flood level and wave action, refer to Section 3.4.

The design of the portion of the intake canal side slopes and wave protection dikes to resist wave action is based on the tables, curves, and method presented in Reference 25.

In general, the following data are the basic assumptions and criteria:

a. Maximum high water (static) 583.7 feet (I.G.L.D.)

b. Maximum wave run-up height 6.6 feet

c. Maximum side slopes 3 horizontal, 1 vertical

Wave breaking layer material:

3-foot random placed angular quarry stone per the following approximate gradation:

Pounds	Percent
4000 to 6000	35
1000 to 4000	42
400 to 1000	10
100 to 400	3
0 to 100	10
	100

6-inch lower layer of 2-inch minus granular material to protect earthen dike under the 3-foot layer. The top of the wave and flood control dikes is protected by a 12-inch graded and compacted granular material.

See Figures 2.4-6. 2.4-7, 2.4-8, and 2.4-9 for plan, section, and description of the canal and its structures and wave protection material.

2.4.2.2.2 Roof Flooding

The roof plumbing systems are designed based on the following criteria:

a. 30-minute rainfall duration

b. 100-year returned period

c. Rainfall intensity 4.5 inches per hour

d. A continuous rainfall

For rainfall intensity, duration, and frequency curves, see Figure 2.3-1. In addition to the above criteria, the design of the system has a factor of safety of 1.2 to 1.7, depending on location and importance of the pipe or structure. The design of the site sewer system is based on the following criteria:

a. 60-minute rainfall duration

b. 25-year returned period

c. Rolling surface with grass, (C = 0.40)

d. Rainfall intensity 2.25 inches per hour

e. Continuous rainfall

All Class I structures have a minimum 2.5-foot parapet at the periphery of their roofs except the service water pump room structure which has no parapet. If for any possible reason rainfall runoff accumulates on roofs, the structures are capable of tolerating this loading condition. The accumulated runoff water loading condition did not control the design of the Class I structure's roof.

2.4.2.2.3 Penetrations Below 585 Feet (I.G.L.D.)

All electrical duct banks with the conduit level at El 575'-0" or lower that enter safety-related buildings from the outside are covered by a membrane (see Figure 2.4-10). This membrane extends to elevation 584'-0" at the upper edges and where applicable, the lower end will be sealed to the building substructure membrane. See Section 3.4.1 for a discussion of flooding of conduits and duct banks.

Penetration M-72 and M-73 are mechanical piping penetrations (located at Column line F-11 and F-10 respectively at elevation 577'-3"). A flexible boot seal will be provided around the pipe sleeve, and it will be sealed with a waterproofing compound. Figure 2.4-11 shows the type of detail which is used for flood protection.

The Service Water Tunnel and two valve rooms are protected up to 570 feet (I.G.L.D.) by a waterproof membrane. The pipe penetrations into the Service Water Tunnel and valve rooms are sealed from in leakage as detailed in Figure 2.4-12. The Service Water Tunnel is located between two Seismic Class I valve rooms. The pipe penetrations into these Seismic Class I structures are sealed as shown in Figure 2.4-12. Therefore, there will be no possibility of flooding the tunnel and valve rooms due to the inleakage during the maximum probable flood.

All electrical duct penetrations below 585 feet (I.G.L.D.) in the service water valve rooms, turbine area, and water treatment area are sealed in accordance with the detail shown in Figure 2.4-13.

There are several doors in the main station at 585 feet (I.G.L.D.) which is above the maximum probable flood elevation.

There are two 7-foot x 2½-foot watertight doors in the intake structure, both at 576.5 feet (I.G.L.D.). The doors are type PRS D3 as manufactured by Presray, Pawling, New York and are designed for a 10-foot head of water with a safety factor of 2.0 (or 20-foot head). The body of the door is constructed of a steel plate not less than 5/8 inches thick. The welding is in accordance with American Welding Society Spec. D1.0. The doors are quick acting, operate from both sides by dual levers, are balanced and aligned, and provide a positive, complete seal. For details, see Figures 2.4-14, 2.4-15, 2.4-16, and 2.4-17.

There is an opening at 585 feet (I.G.L.D.) for stairways from the Turbine Building to the feedwater pump room in the Auxiliary Building. An 8-inch curb is placed around the opening to stop any possible water flow from a pipe rupture in the area. Also, an 8-inch curb is placed in front of doors No. 320 and 323 in front of the high voltage switchgear to prevent any possible water flow due to a pipe rupture.

2.4.2.3 Flood Design Considerations at the Site

For probable maximum rainfall estimates for site drainage, see Subsection 2.3.1.2.1. This value for the 6-hour period is estimated to be 26.7 inches, which is more than 6 times the amount of rainfall expected at the site in 100 years and is based on the hydrometeorological procedures in Reference 34 used in performing the analysis in Reference 60.

The following analysis demonstrates that overflow runoff at the site will not constitute a flood threat to the station:

If for any possible reason the main discharge pipe in the sewer system fails to handle the estimated probable maximum runoff effluent into the river, the runoff water will build up in the system and on the ground around the station. As a conservative assumption, we assumed that the pipe failed at the beginning of the rainfall, and the total volume storage capability of the system (pipes, manholes, catch basins and ditches) is ignored. The average invert elevation of manholes and catch basins is 582 feet (I.G.L.D.). The high-point elevation of roads and ground around the buildings is 584.0 feet (I.G.L.D.). With 26.7 inches estimated runoff, theoretically water could build up to 584.5 feet (I.G.L.D.), but runoff water will overflow to the existing marshes which are at an approximate elevation of 570.0 to 575.0 feet (I.G.L.D.).

Since all the structures are protected against water buildup and flooding up to 585.0 feet (I.G.L.D.), there will be no threat to the structure for this probable maximum buildup runoff water. All the roof drains are gravity flow and located at higher elevations, and rainfall will flow to the Sewer System. As previously mentioned, roofs of Class I structures are protected against any possible built up water and can tolerate this probable maximum of 26.7-inch intensity.

The Shield Building roof has a 3-foot, 3-inch parapet in the area above the low point of the dome. There are no penetrations in this area. Four 4-inch diameter downspouts are provided for roof drains.

The Auxiliary Building roofs have a minimum 2-foot, 6-inch parapet above the nominal elevation. Several penetrations exist in this area. Each of these penetrations is provided with a minimum curb height of 1 foot, 6 inches around the opening. In order to prevent the maximum build-up height of runoff water from overflowing these curbs, an auxiliary drain system is provided. This system is composed of horizontal drain pipes in the parapets. The horizontal drain pipes are in addition to the primary roof drains. The invert elevation of these horizontal pipes is approximately a minimum of 12 inches below the top of the curbs. The horizontal drain pipes are designed to drain the maximum probable rainfall should all the roof drains become stopped.

Therefore, all safety-related systems and components housed in the containment area and Auxiliary Building are protected against flooding from roof penetrations due to rainfall.

There are no penetrations in the roof of the Service Water Valve Room Number 1 (adjacent to the intake structure). The Service Water Pump Room (in the intake structure) ventilation system penetrates the roof into two penthouses. The penthouses are mounted on four inch curbs to prevent rainwater from flowing into the pump room. The periphery of the roof, which has no parapet, allows water to run off the roof instead of accumulating.

2.4.3 Probable Maximum Flood (PMF) on Streams and Rivers

The Toussaint River empties into Lake Erie southeast of the station's site and this stream flows close to the south of the site. This stream becomes Toussaint Creek about six miles upstream from its mouth.

The headwaters of the Toussaint Creek have a maximum elevation of about 670 feet (I.G.L.D.). This stream has a drainage area of about 143 square miles and an average slope of about two feet per mile. There are no dams on this stream. The lower six miles of the stream are much wider than the remainder, and as a result, its level in this wider section is controlled by the level of Lake Erie. In this wider section, it flows at approximately the level of Lake Erie.

The U.S. Geological Survey has operated a spot check stream flow station at a point about 1-1/4 miles west of Limestone, Ohio. The Toussaint Creek at this flow station drains about one-half of the total drainage area of the total stream flow shown in Table 2.4-2. During peak periods of precipitation, the flows in this stream will be higher.

2.4.3.1 Probable Maximum Precipitation (PMP)

Using standard hydrometeorological procedures in Reference 34, a 24-hour, 200-square mile PMP of 22.8 inches was derived. Based upon this value, the PMP for an area of 143 square miles for various durations was obtained. The adjusted PMP of 24 hours duration is 24.1 inches (Reference 61). The selected maximized steam precipitation distribution in Table 2.4-3 was obtained by following procedures outlined in References 35 and 36.

2.4.3.2 Precipitation Losses

Precipitation losses include infiltration, evaporation, depression storage, interruption, and transpiration. These are defined as "losses" in the sense that they do not contribute to direct surface runoff. An assumption of no initial loss was made such that the selected PMP would generate maximum runoff within the watershed. Since the surface soils in the Toussaint drainage area are largely derived from Wisconsin glacial till and lacustrine clays which have a low-to-moderate infiltration capacity, a minimum infiltration loss rate of 0.035 inches per hour was selected as given in References 32, 37, and 38. The estimated precipitation losses and runoff are shown in Table 2.4-4.

2.4.3.3 Runoff Model

Because the Toussaint is ungauged, except for spot check readings, a total flood runoff hydrograph was developed utilizing Franklin F. Snyder's method of synthetic unit hydrograph derivation as given in Reference 38. The unit hydrograph of the watershed was determined near the mouth of the river south of the site. The Toussaint River runoff model is provided in Reference 62. It was developed from the following equations:

$$t_p = C_t (L L_c)0.3$$

$$t_r - \frac{t_p}{5.5}$$

$$t_p R = t_p + \frac{t_R - t_r}{4}$$

where t_p = basin lag, defined as the time from the centroid of rainfall excess to the unit hydrograph peak, in hours

C_t = coefficient representing variations of watershed slopes and storage (C_t = 2.0 assumed for basic area)

L = length of the main stream channel, in miles, (L=40.0 miles for the drainage area under study)

L_c = length along main channel to a point opposite the watershed centroid, in miles (L_c = 22.5 miles for drainage area under study)

t_r = duration of unit rainfall excess, in hours

q_p = unit hydrograph peak, in cubic feet per second

C_p = coefficient accounting for flood wave and storage conditions (C_p = 0.625 assumed for basin area)

A = drainage area, in square miles

t_pR = modified lag time, in hours

t_R = desired unit hydrograph duration, in hours

The unit hydrograph is shown in Figure 2.4-18. It is reasonably conservative in view of floods on other streams in the area as given in Reference 41 and precipitation pattern.

2.4.3.4 Probable Maximum Flood (PMF) Flow

The PMF flow (Figure 2.4-19) was derived from the selected PMP (Subsection 2.4.3.1). The calculated peak flow is 81,970 cfs with a time lag of 31.5 hours (Reference 62). There are no dams or other regulating hydraulic structures of the Toussaint which affect the hydrograph. The PMF stream-course response cannot be assessed since the Toussaint is only spot checked.

2.4.3.5 Water Level Determinations

The Toussaint water level near the site is normally controlled by Lake Erie. Due to the flat topography near the site area, and considering normal water levels of Lake Erie, the PMF discharge would backup at the river’s mouth, causing inundation of areas adjacent to the channel bed, and occupy a floodplain area of considerable extent. These flood waters would discharge to Lake Erie both upstream and downstream of the site area and also extend south to and along Rusha Creek, contributing to backwater effects in that drainage area. The Highway 2 bridge south of the plant area is at such a low elevation that flow of the PMF would inundate it along with much of the roadway near the site. During the PMF, flow velocities at the bridge waterway would be rather low, and the bridge itself would not produce the particular backwater problems. After recession of the storm, much of the floodwaters would remain as depression storage.

Assuming the very conservative assumption that none of the water discharged to Lake Erie while the Toussaint remained at normal water level near its mouth, the high water level due to the PMF hypothetically “dammed up” at that point would occur below elevation 579 feet (I.G.L.D.), with flood plain effects far upstream of the plant area (Reference 62). The storage would be about 176,000 acre-feet. Flood plain areas are represented in Figure 2.4-20.

2.4.3.6 Coincident Wind Tide Activity

The maximum Lake Erie static water level due to wind tides (Subsection 2.4.2.2) is 583.7 feet (I.G.L.D.). At this maximum static water level, the lake water would extend more than five miles upstream from the station site. Thus, the PMF water from the Toussaint would be dissipated to the lake prior to reaching the station site and would not increase the static water level at the site.

2.4.4 Potential Dam Failures

2.4.5 Probable Maximum Surge Flooding

A description of the Probable Maximum Meteorological Event (PMME) for the station and its effect on wind tides and seiches at the station is given in Subsection 2.4.2.2 and the following subsections.

2.4.5.1 Comparison of Lake Erie Storms with PMME

Safety considerations for the Davis-Besse site given in Subsection 2.4.2.2 require that the PMME will occur during the design maximum monthly mean lake level of 573.4 feet (I.G.L.D.). The November 14, 1972, storm occurred during a time when the Toledo mean lake level was at 572.10 feet (I.G.L.D.). The April 9, 1973, storm occurred when Toledo mean lake level was at 573.36 feet (I.G.L.D.).

The PMME requires that the maximum wind speed reach 100 mph for at least 10 minutes and be preceded and followed by 6 hours of 70 mph average winds. During this 12 hour period, winds must be from the ENE. The wind speed during the November 1972 storm is calculated to have reached a maximum over water velocity of 33 mph. These winds were preceded and followed by winds of variable velocity which ranged from 15 to 20 mph. During this time wind direction was also variable and ranged from N to ENE.

The maximum wind tide associated with the PMME would produce a rise of approximately 11.6 feet at Toledo above the mean lake level and a rise of 9.3 feet at the Davis-Besse site with an additional one-foot rise due to a transverse seiche. For the maximum design considerations, these surge rises were assumed to occur at a time when the monthly mean lake level was 4.8 feet above datum.

The November 1972 storm produced a wind tide at Toledo of 3.8 feet above the November mean lake level or 7.8 feet below that associated with the PMME. The November storm occurred at a time when the mean lake level was at a record high for November but 1.3 feet below that postulated for the PMME. The April 1973 storm produced a wind tide at Toledo of 3.31 feet above the April mean lake level or 8.29 feet below that associated with the PMME. The April storm occurred at a time when the mean lake level was at a record high for April.

The April 1973 storm exemplifies the conservativeness of the PMME storm model. It shows that the sequence of events necessary to reproduce the water levels required for the PMME would

be very unusual indeed, and if the sequence ever was duplicated, the power station is designed to withstand the resulting wind tide wave action, and flooding.

The analysis and comparison of the November and April storms with the PMME indicates further that the offshore islands cannot contribute to the waves that ultimately strike the elevated areas around the power plant. The controlling parameter is water depth, which precludes additional wave buildup due to increased fetch length. See Subsection 2.4.5.2 for additional analysis of the effect of the offshore islands of wave height.

In addition to the November 1972 and April 1973, other storms producing high lake levels at Toledo, Ohio have been studied. Four storms are summarized in Table 2.4-5 which shows the calculated peak wind setup elevations using Platzman's model and the measured peak setup at Toledo, Ohio. Other storms of lesser intensity have been checked, but are not included in Table 2.4-5.

2.4.5.2 Estimate of PMME Wave Action Generated in Eastern Basin of Lake Erie and Transmitted into the Western Basin

The height and period of wind-generated waves in deep water are a function of the wind fetch, speed, and duration. In shallow water, the depth with an allowance for the increase (or decrease) due to wind setup is an additional factor.

The fetch is the length of water over which the wind acts in the process of wave generation. The fetch may be limited by the dimension of the wind field or by the dimension of the body of water. For island bodies of water where the dimensions of the body itself are limiting, the fetch width as well as fetch length must be considered. The total fetch along the longitudinal axis of the lake to the islands off Pelee Point is about 200 nautical miles. The effective fetch length (corrected for fetch width) would be somewhat less, but the corresponding reduction in generated wave height would be negligible as the PMME waves reaching the islands are not fetch limited.

The wind speed used for these calculations is 100 miles per hour, which corresponds to the maximum PMME wind speed over the lake. The average wind speed over the lake's longitudinal axis at the time of maximum winds is only 89 miles per hour. Therefore, the generated wave height is conservative.

The water depth over the fetch used for these calculations is the sum of the average depth indicated on the nautical chart of the lake, (referred to low water datum), the maximum mean monthly lake level, and the PMME wind setup. In the western basin, a total water depth of 40 feet was used, and in the portion of the lake east of the islands 65 feet was used. The effect of bottom friction on wave generation and decay is considerable in shallow water. The effect has been considered in the calculation of generated wave heights. However, the effect of wave decay has been neglected, providing conservative results in the shallow portion of the lake near the islands.

The maximum significant wave height that would be generated east of the islands is 18 feet with a period of 7.0 seconds. These waves would become unstable and break when the water depth is reduced to 23 feet as the waves run over a gradually shoaling bottom.

These waves could be transmitted into the western basin of the lake through the inter-island channels as shown in Figure 2.1-1. However, the islands still serve as effective obstructions in sheltering the Davis-Besse site from the direct attack of these waves. The wave energy

entering the western basin through the islands shown in Figure 2.1-1 can undergo diffraction and refraction into the wave shadow created by the islands and thereby approach the site. Inspection of the bathymetry of the area indicates that the waves which approach the site from the generating area east of the islands and undergo diffraction and refraction will be severely attenuated. While regeneration of these waves over the fetch between the islands and the site is possible, the regenerated waves will be comparable to those waves generated within the western basin itself over the critical NNE fetch.

The critical fetch for PMME wave generation in the western basin is from the NNE direction with an effective length of 23 nautical miles. Not only is this the longest available fetch, but it also provides the greatest depths. Furthermore, waves approaching the shoreline at the Davis-Besse site from this direction are subjected to little or no reduction in wave height as a result of refraction. The highest significant wave that would be generated is 11.7 feet with a period of 5.2 seconds. The maximum wave height that would be generated is 21.9 feet with a maximum period of 6.5 seconds. This wave would become unstable and break when the water depth is reduced to 28 feet. This depth would be reached approximately 3.5 nautical miles from the existing shoreline at the Davis-Besse site, when allowance is made for the increased depth due to high PMME lake levels. Although waves larger than 22 feet could be generated east of the islands, should they succeed in entering the western basin of the lake they would break even farther offshore. The maximum wave height that can reach the beach berm during the time of critical wave attack is 10.7 feet. Therefore, regardless of the area of initial generation of the wave and its subsequent modification during its travel to the beach fronting the site, it will break and reform at a lower height and be successively reduced in height. The available depth of water over the gradually sloping beach that fronts the site is responsible for limiting the maximum wave height that can approach the Davis-Besse site to 10.7 feet.

2.4.5.3 Estimates of Wave Action On and Along the Intake Canal and Its End Structures and Potential for Wave-Induced Resonance in the Intake Canal

The intake canal terminates at the lake shore, and there is no structure on the lake side of this canal. See Figure 2.2-2. As indicated on Figure 2.2-2, a Seismic Class II 96-inch pipe buried under the lake bottom connects the shore intake canal to an offshore Seismic Class II intake crib, which is located approximately 3300 feet from the shoreline. The intake crib is buried under the bottom of the lake except for the screen structure, which is at 561.85 feet (I.G.L.D.) (the top of the structure). There is no possibility of any severe wave action on this structure and the buried pipe during probable maximum surge flooding, which is at 583.7 feet (I.G.L.D.) (see Subsection 2.4.2.2).

The intake canal has Seismic Class II dikes at each side and at the lake side. The approximate 3000-foot-long dikes, from lake side, have an elevation of 579 feet (I.G.L.D.) which is approximately 4.7 feet below the static water level associated with the PMME. The probable maximum wave height in the marsh areas is 8.5 feet (see Subsection 2.4.2.2), therefore, there will be no significant wave effect on these dikes. The top elevations of the additional 700-foot Seismic Class I dikes, at the station end of the canal, are at El. 582.0 and 591.0 (I.G.L.D.). From Station 0+00 to approximately 5+50, the inboard dikes are covered with wave-control material (see Subsection 2.4.2.2 and Figures 2.4-6 and 2.4-8). The calculated maximum wave run-up at this area is 6.6 feet, and the dikes are designed for the wave action and impact of the waves.

The remaining inside slopes of the intake canal are also covered with wave-control material; however, this area is for protection of the normal lake level fluctuations and is not associated with the Probable Maximum Meteorological Event.

The magnitude of wave forces against the intake structure was calculated in accordance with "Design and Construction of Ports and Marine Structures" by A.D. Quinn and "Shore Protection Planning and Design" by U.S. Army Coastal Engineering Research Center, Tech. Report-4. The front wall of the intake structure (see Figure 2.4-21 Section A) which is against the wave action is 2 feet 0 inches thick reinforced concrete and is designed to withstand static and dynamic forces due to waves and wave run-up. The Service Water pumps and other Class I equipment are housed in a water tight room as shown on Figure 2.4-21. This room is not exposed to direct wave action and, therefore, is not affected by forces induced by waves and wave run-up.

While evaluating wave effects on the intake structure, no consideration was given to the dike at the lake intercept of the intake canal. However, as explained in Section 2.4, the larger waves generated in open lake will never reach to the intake structure as they will break away in the shallow water. It is, therefore, concluded that an assumed loss of this dike will not adversely affect the structural integrity of the intake structure.

Based on the geometry of the intake canal forebay, low elevation of the Seismic Class II earthen dikes at 579.0 feet (I.G.L.D.), the Seismic Class I wave-control dikes, and the wave-control concrete walls at the face of the intake structure, it is concluded that there is no potential for wave-induced resonance in the intake canal. Generally, wave-induced resonance can become critical in the design of harbors, particularly if the sea walls or break walls are vertical and have highly reflective surfaces. At the Davis-Besse power station, these conditions do not exist and there is no resonance problem due to wave activities.

2.4.6 Tsunami Flooding

Wind-generating waves and run-up were determined to be as high as five feet above the elevation of the top of the basin wall for the cooling tower.

In order to determine the dynamic effect of the wave in terms of static force, an equivalent static force of 5,000 pounds per square foot of exposed area for the height of the wave has been determined.

The analysis took into account the effects of such a wave on those elements of the structure which would receive the impact of the wave, namely the diagonals supporting the veil, the column bents supporting the fill structure, and the prestressed concrete pipe carrying the hot water supply.

The assumption was made, in checking the strength of the diagonals, that the wave could achieve its maximum impact force coincident with the full effect of a design wind force. Using this maximum operating condition, it was found that the reinforcing and the concrete in the diagonal would certainly be overstressed and probably would cause some cracking. It is clear, however, that there would be no failure and that the veil diagonals would continue to carry their design load. It would be advisable, if such a loading condition should occur, to make a thorough field check of the condition of the diagonals and make a recommendation for remedial repair work. It should be borne in mind that the maximum wind force and the effect of wave action would affect only one or two columns in a maximum condition.

The force of the wave would obviously be maximum at the diagonals and would tend to be somewhat broken up by the diagonals themselves, but still would maintain a large percentage of the force at the time of impact on the fill support column bents. The check was made of the

column bents assuming the full force, and, here again, the result was an indication that there would be some overstressing and resultant cracking, but no collapse. The impact force would be distributed to the slab where there is adequate shear resistance and to the intersection of the top of the frames at the veil where calculations indicate no serious effect would be realized on the veil. The impact force would probably jar loose a small number of the fill sheets, but it is our opinion that the cooling capability of the heat exchanger would not be substantially affected. Since the fill support structure is not affected to any great extent by wind forces, the only loading to be considered is the dead load of the fill structure plus the effect of the wave action. It would appear that local cracking of the column bents would occur and such cracking should be checked for possible remedial repair, but that the fill structure in general should withstand the wave action without serious impairment of its function.

The hot water inlet pipes are so situated that by the time a wave could impact on the piping its force would be substantially reduced. In any case, the pipe is supported on saddles, and the most serious problem would be the tendency of the lateral force of the wave to lift the pipe off its saddles and rupture the pipe in this way. We can assume that the wave would hit during an operating condition in which case the pipe would be filled with water and the calculations indicate that the vertical load of the water plus the pipe would more than offset the effect of the wave force and would inhibit the possibility of so displacing the pipe. The pipe has more than adequate strength to resist the lateral force by its own beam action spanning between saddles. We, therefore, conclude that no serious problem would result from the wave action on the piping.

Based on our analysis, we conclude that the design wave which may be expected under extreme conditions would not impair the cooling function of the cooling tower.

2.4.7 Ice Flooding

Flooding of the safety-related structures and equipment at the Davis-Besse site due to ice jams in the Toussaint River is not credible. The probable maximum rainfall in this stream's drainage area would cause the water to rise at the station to 580 feet (I.G.L.D.) if none of the water reached Lake Erie. However, the station will be protected from flooding up to an elevation of 585 feet (LG.L.D).

Floating fields of lake ice may exert a major pressure on near-shore structures, when driven by a strong wind or current, by piling up in large ice packs against the obstructions. This condition must be given special attention in the design of small isolated structures. However, because of the inherent flexibility of the ice field, the pressures exerted on large open structures are not as great as would be caused by a solid ice sheet in a confined area.

Ice formations may at times cause damage to shorelines in local areas, but their net effects are largely beneficial. Spray thrown up by wind and wave action during the winter may freeze on the banks and structures along the shore, covering them with a protective layer of ice. Ice piled on shore by wind and wave action does not, in general, cause serious damage to beaches, bulkheads, or protective riprap, and generally provides additional protection against damage from the severe winter storm waves. Ice often has a definite effect on impoundment of littoral drift. Updrift source material is less erodible in the frozen state and windrowed ice acts as a barrier to shoreward moving wave energy; therefore, the quantity of material reaching an impounding structure is reduced. During the winter of 1951-52 it was estimated that ice caused a reduction in rate of impoundment of 40 to 50 percent at the Fort Sheridan, Illinois groin system (ref. 25).

The Lake Erie shoreline in the vicinity of the station site experiences occasional buildup of ice ridges from strong northeasterly wind action during late winter periods after the lake has experienced extensive ice cover. These ice ridges normally form in shallow water several hundred feet from the shoreline and during periods of high water could form close to the beach ridge which is at 575 feet (I.G.L.D.). Under extreme high water conditions, any ice that could cross the beach front area into the marsh area would be stopped by the wave protection dike along the north, east, and a small portion along the south sides of the station building area.

Ice flooding will be of no concern at the Davis-Besse site. The elevation of the plant structures is above the level of normal lake ice formations. Should the PMME occur during the winter months, ice formation over the plant site area from residual flood waters would be of no concern as the plant is adequately designed to preclude flooding of any type. Category I wave protection dikes are designed to withstand the impact of ice floes which might possibly be driven the entire 3,000 feet from the normal shoreline to the dike system which guards the station.

The formation and subsequent thawing of ice on Lake Erie has been studied and it was found that the morphology of the lake basin has a definite effect on ice formation. The Lake Erie basin is divided into three principal sub basins: a small, shallow basin at the west end which borders the site and is partially restricted by a chain of barrier beaches and islands; a flat, unrestricted and rather shallow basin in the center; and a small, relatively deep eastern basin.

Temperature within the three basins usually ranges between 75°F (24°C) and 39°F (4°C). Complete mixing of the western basin in late fall causes it to cool more rapidly than the central and eastern basins. The first lake ice is reported most often in the western basin in late December.

Relative winter severity is described by the number of freezing degree-days, that is, the number of days having a mean temperature of one degree below 32°F (0°C). A day having a mean temperature of 22°F (-12°C) would equal ten (10) freezing degree-days. The most recent information available concerns the winter of 1976-1977 which is classified as a severe winter due to its great number of freezing degree-days. An average Lake Erie winter produces about 290 freezing degree-days. The winter of 1976-1977 produced over 700 freezing degree-days.

During the 1969-70 winter, which produced nearly 700 freezing degree-days, ice was first reported on Lake Erie at the Enrico Fermi Power Plant water level gauge on December 4, 1969. This ice cover in the western basin was unusually early and created problems for shipping. By February 17, 1970 the lake was estimated to be 95 percent ice covered. Most areas had thick concentrations of winter ice which carried a heavy, drifted snow cover. The entire western basin was ice covered and the rafted and jammed areas were restricted to the eastern basin near the Niagara River outlet. By March 24 most of the ice cover had disintegrated, leaving many areas ice free. The western basin near the site was clear except for some drift ice and shore ice. Even though this was an unusually severe winter, this type of breakup exhibits the characteristic ice distribution pattern for Lake Erie; drift ice concentrating along the southern and eastern shores leaving the northern and western shores open.

2.4.8 <u>Cooling Water Canals and Reservoirs</u>

2.4.8.1 Canals

There are two canals associated with the cooling water systems: the intake canal and the condenser cooling water open channel. The intake canal from Lake Erie to the intake structure serves as a barge facility during construction, provides makeup to the Circulating Water System

and other systems, and provides water to the Service Water System and the Fire Protection System. The forebay of this canal, adjacent to the intake structure, is designed as Seismic Class I and is designed for the Probable Maximum Flood (PMF). For the design basis and details of the intake canal see Section 3.4. The Probable Maximum Flood Water is elevation 583.7 feet (I.G.L.D.), and the top of the flood control dike is elevation 591 feet (I.G.L.D.). A freeboard of 7.3 feet will resist any wave action and run-up from Lake Erie.

The Condenser Cooling Water System is a closed system. The open channel (cold line) between the cooling tower and the pump house is a Class II structure. This channel is designed to withstand the hydrodynamic effects of the closed Condenser Cooling Water System, rather than any wave action or PMF requirements.

2.4.8.2 Reservoirs

There is no reservoir in the Condenser Cooling Water System. The intake canal forebay is designed as Seismic Class I, and will be used as a heat sink reservoir during accident and Maximum Possible (larger) Earthquake. For a description of the slope stability and seismic analysis of the impending earthen dikes of the Class I portion of the intake canal see Subsections 2.5.5 and 3.7.2.10, respectively.

2.4.9 Channel Diversion

The water flow through Lake Erie and subsequent mean lake elevation varies on an annual and long-term cyclical basis due to rainfall variations in the Great Lakes drainage basin. The mean lake level, however, is not subject to variations due to diversion or source cutoffs.

2.4.10 Flooding Protection Requirements

For complete detail and design basis of the probable maximum surge flooding and the flood water level design criteria, see Subsection 2.4.5 and Section 3.4, respectively.

2.4.11 Low Water Consideration

2.4.11.1 Low Flow in Rivers and Streams

Since the station is taking all of its cooling water requirements from Lake Erie, low flows in the Toussaint River will not affect the station's operation.

2.4.11.2 Low Water Resulting from Surges

The static water levels in the western basin of Lake Erie are subject to long term and annual cyclic variations in the mean monthly level from the low water datum and to short period variations in the daily level from the monthly mean level due to wind tides and seiches. This subject of lake levels has been investigated and the results of this investigation are summarized in this section.

The low water datum of Lake Erie is 568.6 feet (I.G.L.D). The maximum variations in the mean monthly level are 4.2 feet above datum and 1.2 feet below datum for the 110-year period that data has been collected. A probable maximum variation of 4.8 feet above and 1.5 feet below datum has been used at Davis-Besse.

The short period variations in the daily level from the monthly mean level are due to both a lengthwise wind tide which produces the greatest disturbance of water level and a transverse seiche in the west end of Lake Erie which can oscillate between the northern and southern shores. The recorded maximum transverse seiche has been 0.8 feet and a probable maximum value of 1.0 feet at the site is assumed. A uninodal lengthwise seiche on Lake Erie can contribute some increment to the wind tide but the maximum amplitude of the uninodal seiche cannot coincide with the maximum wind tide.

A similar Probable Maximum Meteorological Event was used to determine the maximum drop in lake level due to wind tides that was used in Subsection 2.4.2.2 to calculate maximum high water levels. This meteorological event would have maximum WSW winds at any one location of 100 miles per hour for a 10-minute period and the wind speed would exceed 70 miles per hour during the 6-hour period both before and after the maximum wind speed. The procedure developed by Platzman (ref. 3) was applied to this storm to determine the maximum wind tide fall at Toledo. Since the Davis-Besse site is located about 80% of the way from the wind tide node (point in lake where no wind tide change in lake level occurs) to Toledo, wind tide variations at the Davis-Besse site were reduced by 20% from Toledo wind tides. This procedure gives a maximum wind tide drop with WSW winds of 9.3 feet.

The extreme low level wind tide of 9.3 feet could occur at this time when the monthly mean lake level was at the long-term low probable level of 1.5 feet below low water datum, and when the transverse sieche was removing one foot of lake elevation. The total would be 11.8 feet below datum or a probable extreme low water level of 556.8 feet (I.G.L.D.).

2.4.11.3 Historical Low Water

The lowest static water level (no wind tide effect) on Lake Erie recorded by the U.S. Department of Commerce, National Ocean survey was 567.4 feet (I.G.L.D.) in February, 1936. The lowest water level (including wind tide effect) recorded at Toledo by the U.S. Army Corps of Engineers was 561.0 feet (I.G.L.D.) on December 27, 1938. The lowest recorded water level at Toledo is 4.2 feet higher than the low water level predicted at the Davis-Besse site during the Maximum Probable Meteorological Event described in Subsection 2.4.11.2.

2.4.11.4 Future Control

There are no planned dams in the Great Lakes drainage area that can significantly change the extreme low water levels of Lake Erie.

2.4.11.5 Station Requirements

The station's minimum cooling water requirements in the winter, when the low annual lake level normally occurs, is about 15,000 gallons per minute for the cooling tower makeup flow. The station's service water is returned to the intake canal at the intake structure to prevent freezing during the winter, and is used as cooling tower makeup at other times during the year.

The intake canal is dredged to an elevation of 556.3 feet (I.G.L.D.) at the lake end. This intake canal is terminated with a diked closure at the original shoreline. This diked closure is built up to an Elevation of 579 feet (I.G.L.D.) which is about four feet above the remainder of the beach dike level.

To supply water to this closed intake canal, a submerged 96-inch conduit is extended into the lake approximately 3,300 feet to an intake crib shown on Figure 2.2-2. Water flows down

through slots in the top of this intake crib. The top of this crib was built at an elevation of 561.85 feet (I.G.L.D.) and the intake canal will be completely cut off from the lake if the lake water level falls below this elevation.

2.4.11.6 Dependability Requirements

The station can obtain its minimum water requirement of 15,000 gallons per minute from the lake at any lake water level above 562.1 feet (I.G.L.D.). At any water level below this, the station cannot be assured of obtaining sufficient lake water for continuous rated power generation.

Should the water level of the intake canal fall below the 562 foot (I.G.L.D.) level, shutdown of the station will be initiated as described in the Technical Specifications. All power generation would be stopped, all cooling tower blowdown would be shut off, and all service water would be returned to the Seismic Class I forebay area. Figures 2.4-21, 2.4-6, 2.4-7, 2.4-8, and 2.4-9 show the design of the intake structure and the forebay area of the intake canal. All decay heat would be removed initially using either the turbine bypass system, condenser and cooling tower or the atmospheric vent valves. As the decay heat load decreases the decay heat coolers would be placed in service and the Service Water System would use the forebay area as a cooling pond. The design precludes any hydraulic short circuiting since all the nonessential water requirements can be isolated by proper valving that is incorporated in the design.

During the maximum probable low water conditions described in Subsection 2.4.11.2, the intake canal would be completely cut off from the lake; however, due to the design of the intake system in the lake the intake canal cannot be drained below 560 feet (I.G.L.D.). There is, however, sufficient water impounded in the deeper Class I excavation at the intake structure forebay area below 560 feet (I.G.L.D.) to provide for a safe and orderly shutdown of the station and to provide sufficient cooling water to maintain the station in safe condition for at least 30 days. The Service Water pumps, which supply cooling water to the engineered safety features, require a minimum water elevation in the forebay area of 554 feet (I.G.L.D.) for this postulated condition. Low water due to a lake surge condition is a short-term condition of less than approximately 12 hours duration.

The heat dissipation and inventory capability of the intake canal under long-term low water condition resulting from a loss of lake supply is given in Subsection 9.2.5.1.

2.4.12 <u>Environmental Acceptance of Effluents</u>

The hydrologic features of the site and site area, together with the related station design features, enable the environment to accept normal, inadvertent or accidental releases of radioactive liquid effluents without undue risk to the general public.

Normal releases of radioactive effluents, which will be at an extremely low level, are discharged to the lake with the cooling tower blowdown and other station liquid effluents. The design and arrangement of the discharge structure which provides for rapid dilution with adjacent waters and the subsequent additional dilution resulting from natural lake currents provides large dilution factors for release from the station. The distance from the station of the nearest potable water intakes discussed in Subsection 2.4.1.2.4, and the large dilution factors for releases, provides assurance that any type of credible release can be accepted by the environment without undue risk.

The design of the station structures and fill arrangement provides an unlikely condition that any accidental release of radioactive liquids would be introduced into the groundwater aquifer. In addition, there is an almost total lack of use of the groundwater in the site area for human consumption as discussed in Subsection 2.4.13.2.

Dispersion of material in a saturated porous medium can be described by Fickian Diffusion (ref. 3). Sutton has provided solutions to the Fickian diffusion equations for instantaneous and continuous point source releases in the form of the Gaussian function (ref. 4).

Continuous Source:

$$\frac{C(x,y,z)}{C_o} = \frac{Q}{2\pi\pi_y(x)\sigma_z(x)v}\exp\left[-\left[\frac{Y^2}{2\sigma_y(x)^2}+\frac{Z^2}{2\sigma_z(x)^2}\right]\right]$$

Instantaneous Source:

(2)

$$\frac{C(x,y,z,t)}{C_o} = \frac{V}{\sqrt{2}\pi^{3/2}\sigma_x(x)\sigma_y(x)\sigma_z(x)}\text{EXP}\ -\left[\left[\frac{(x-vt)^2}{2\sigma_x(x)}+\frac{y^2}{2\sigma_y(x)}+\frac{x}{2\sigma_z(x)}\right]\right]$$

Where:

$C(x,y,z)/C_o$	=	Concentration dilution factor at point x,y,z
Q	=	Rate of flow into the aquifer of the discharge initial concentration C_o - cfs
V	=	Volume of discharge of initial concentration C_o - ft^3
t	=	time – sec
$\sigma_x(x),\sigma_y(x),\sigma_z(x)$	=	Dispersion parameters
v	=	Darcy velocity of aquifer - ft/sec

The dispersion parameters $\sigma_i(x)$ may be related to the diffusion coefficients K_i by[5]:

$$\sigma_i(x)^2 = 2K_i\frac{x}{v} \tag{3}$$

The coefficients are a function of the porous media and have been expressed by Harleman (ref. 6).

$$K_x = 0.9\,\nu\left[\frac{v}{n}\frac{d}{\nu}\right]^{1.2} \tag{4}$$

$$K_y = 0.087\,\nu\left[\frac{v}{n}\frac{d}{\nu}\right]^{0.75} \tag{5}$$

Where:

- υ = Viscosity, ft²/sec
- n = Porosity
- d = Characteristics pore diameter – ft
- v = Darcy velocity, ft/sec

The centerline dilution factor (for a continuous source) and the center dilution factor (for an instantaneous source) as a function of distance (x) can be written from the above equation as:

$$DF_c = \frac{Q}{1.1\nu^{0.25} v^{0.75} \left[\frac{d}{n}\right]^{0.75} (X + x_v)} \qquad (6)$$

$$DF_I = \frac{V}{1.84^{\,0.15}\, v^{-0.15} \left[\frac{d}{n}\right]^{1.35} (x + x_v)^{1.5}} \qquad (7)$$

Where X_v is a virtual source distance which accounts for the initial Volume of the source. Its value is found by setting DF = 1, X = 0 and solving for X_v.

Porosity and characteristic pore diameter have not been determined for the bedrock aquifer material. However, typical ranges for limestones are 0.01 to 0.1 for porosity[5] and 1 mm to 10 mm for diameter. Using the extreme range of the ratio, d/n, the following expressions are obtained for the range of dilution factors.

$$DF_c = \frac{31Q}{v^{0.75} + (X - X_v)} \quad \text{to} \quad \frac{0.98Q}{v^{0.75}(X + X_v)} \qquad (8)$$

$$DF_i = \frac{10.4V\ v^{0.15}}{(X + X_v)^{1.5}} \quad \text{to} \quad \frac{0.11V\ v^{0.15}}{(X + X_v)^{1.5}} \qquad (9)$$

The above equations can be used to estimate the dilution factors for material that is discharged on site and infiltrates to aquifer. However, the predominant direction of regional groundwater flow is to the northwest toward Lake Erie (Subsection 2.4.13.2.3). Groundwater level observations in Woodville Township, Sandusky County (18 miles southwest of site) during the period 1947 to 1969 indicate that the piezometric level has not fallen below 607 feet MSL. Thus, a gradient toward the lake of approximately 1.5 feet/mile was present during this extreme condition and regional groundwater reversal did not occur. However, if a reversal would occur, the following range of dilution factors would be expected if the gradient was equal to the present one, but in a reverse or normal direction (along shore).

For:

$v = 1.24 \times 10^{-7}$ ft/sec

$Q = 2.2 \times 10^{-3}$ cfs

$V = 267$ ft^3

Source	Distance - feet			
	100	1000	5000	50,000
DF_C Min	0.99	0.91	0.67	0.17
Max	0.76	0.24	0.06	6.4×10^{-3}
DF_I Min	0.15	7.6×10^{-3}	7.1×10^{-4}	2.2×10^{-5}
Max	3.1×10^{-4}	1.6×10^{-5}	1.5×10^{-6}	4.6×10^{-8}

Dispersion of material that enters the groundwater from Lake Erie recharge is expected to be minimal since the area of recharge (lake bottom) is anticipated to be large. Therefore, the dilution afforded by dispersion in the lake would be applicable. Dilution values are presented in Subsection 11.2.7.2.

2.4.13 Groundwater

2.4.13.1 General Description and On-Site Use of Groundwater

2.4.13.1.1 Introduction

Information regarding groundwater conditions was obtained by (1) reviewing the existing literature concerning groundwater conditions of the region and the site locality, (2) interviewing representatives of the Department of Natural Resources of the State of Ohio and the Water Resources Division of the United States Geological Survey, and (3) studying 32 logs of water wells existing near the site locality. In addition, 18 water well owners located within approximately two miles of the site were interviewed.

2.4.13.1.2 Regional Groundwater Aquifers

The sequence of sedimentary formations beneath the Portage River basin contains the principal regional bedrock aquifers of northwest Ohio. Water-bearing Silurian and Devonian formations consist of thick layers of limestone and dolomite (described in Section 2C.2 of Appendix 2C). These rock units, although fairly uniform in carbonate composition, are quite dissimilar in physical and hydrologic characteristics. Detailed discussion of occurrence and distribution of groundwater in northwest Ohio is presented in reference 30. According to this report, yields of 150-600 gal/min can be obtained from wells drilled into the carbonate aquifer in the region west of the site (toward Toledo), and as much as 500-1000 gal/min can be obtained from wells drilled into the carbonate aquifer in the region east of the plant site (toward Port Clinton). Approximately 8 miles southwest of the site, the expected yield decreases to 50 to 200 gal/min. Figure 2.4-22 shows the general yields that may be expected for wells developed in the carbonate aquifer in the site region.

Groundwater in the carbonate aquifer in northwestern Ohio is related to bedrock structure. Investigations (ref. 27) have shown that wells of greatest yield are located along flanks of the Findlay Arch (see Figure 2.4-22). A few of these high yield wells are drilled into the upper part of the Lockport dolomite: however, most high yield wells extend several tens of feet into carbonate strata overlying the Lockport dolomite. The surface bedrock formation at the site is the Tymochtee formation.

2.4.13.1.3 Groundwater Quality in the Region

Groundwater from carbonate aquifers in the region exhibits rather large ranges in chemical quality. The differences in quality occur not only on an aerial basis within a particular rock formation, but also with depth.

Representative samples of groundwater from specific units within an aquifer probably have not been obtained: most of the wells penetrate two or more formations with several water-producing zones. Overall quality of a pumped well is probably a composite of all water-yielding strata penetrated, and not necessarily of specific formations.

The quality of water of the carbonate aquifer system is strongly influenced by the dolomite composition of the formations. Calcium and sulfate derived from the mineral gypsum (which occurs as a minor impurity) are the predominant dissolved constituents throughout Silurian and Devonian carbonate rocks of northwest Ohio. Solubility of the gypsum is relatively high compared to dolomite or calcite. Throughout large areas of the carbonate aquifer system, calcium and sulfate constitute from 50-90% of the total mineral content in the groundwater. Groundwater in the carbonate aquifer typically contains unusually high concentrations of dissolved solids, total hardness and non- carbonate hardness. Groundwater in the site vicinity is highly mineralized with sulfate concentrations and total hardness exceeding 1000 mg/l (ref. 32).

Hydrogen sulfide H_2S, derived from sulfur-bearing impurities within the aquifer system, is present in most water wells in the site region. Concentrations of H_2S generally increase with depth, and objectionable levels of H_2S are present in most formations. Concentrations exceeding 3 mg/l are common in water from wells in central and eastern Ottawa County (ref.32).

The primary movement of groundwater within the region is through joints and fissures in the carbonate bedrock formation which are connected on an aerial basis. This system of joints and fissures gives a resultant effect of a radially-homogeneous aquifer, except in the immediate vicinity of a pumping well where the flow assumes some of the characteristics of flow in a linear channel.

2.4.13.1.4 Groundwater Quality at the Site

To evaluate groundwater quality in the vicinity of the site, 15 samples of groundwater were taken from wells drilled into the carbonate aquifer, and water samples were taken from borings and wells in the site locality and from the test excavation; see Section 2C.4 of Appendix 2C. Two surface water samples, one from Lake Erie and the other from the Toussaint River, were obtained for comparative purposes. Sampling locations are shown in Figure 2.4-23; results of chemical analyses on these samples are presented in Table 2.4-6.

Results of these chemical analyses indicate that the groundwater is of inferior quality. Characteristics generally include: pH between 6.8 and 8.0; total hardness generally between

1000 and 2500 parts/million; sulfate content between 800 and 1700 parts/million; and total dissolved solids content generally between 2000 and 4500 parts/million.

Small amounts of gas were observed in several of the borings made at the site. In most of these borings, the gas was hydrogen sulfide; however, the presence of methane gas was also confirmed. The presence of hydrogen sulfide gas causes the groundwater to have an objectionable odor.

2.4.13.1.5 Station Use of Groundwater

There are no plans to use groundwater from the site in the operation of the station.

2.4.13.2 Sources

2.4.13.2.1 Present Use of Groundwater

The following information pertains to the use of groundwater and was provided as part of the original safety analysis. Current conditions are not foreseen to differ significantly, however it is not practical to continuously update this section to reflect the current status. Therefore, this section is considered historical.

Regional:

Current regional usage of the carbonate aquifer system is shown in Figure 2.4- 24. Groundwater pumpage on a daily basis is shown by circles of variable diameter representing both industrial and municipal water.

The largest usage of groundwater from a point source within the site region is the Toledo well field, which pumps over 1.0 million gal/day. Within a 25-mile radius of the site in Lucas, Ottawa, and Sandusky Counties, the total groundwater withdrawal for municipal and industrial pumpage is on the order of 3.5 million gal/day. Total quantity of groundwater pumped daily from domestic wells and farm irrigation wells is not known, but is probably a considerable volume on a regional basis. However, at any particular location, they presently represent a small fraction of the total usage and, consequently, would produce little effect on the regional pattern of groundwater movement.

Local:

To evaluate groundwater conditions at the site, a well inventory was conducted for the area within a few miles of the site. In addition, the owners of 18 wells located within 2 miles of the station were interviewed. Locations of water wells within approximately 2 miles to 3 miles of the site are shown on Figure 2.4-23.

Water from wells in the site locality is used primarily for certain domestic sanitary purposes and for farm irrigation. In most instances, as a result of hardness, objectionable odor, and bitter taste, groundwater is not domestically used for washing, cooling, or drinking. Of the 18 wells inspected, 6 are no longer being used, and the remaining 12 are used only intermittently.

2.4.13.2.2 Potential for Groundwater Development

The following information pertains to the use of groundwater and was provided as part of the original safety analysis. Current conditions are not foreseen to differ significantly, however it is

not practical to continuously update this section to reflect the current status. Therefore, this section is considered historical.

Regional:

On a regional basis, potential development of the carbonate aquifer in northwestern Ohio far exceeds present demands for groundwater development in areas outside of the existing pumping centers.

The projected water demands for communities within 20 miles of the site indicates that the total demand for water in the year 2006 in all communities will exceed total present well capacity of the communities by approximately 2 million gal/day (ref. 32). Assuming the increase in water supply for each community is supplied by pumping from additional wells drilled into the carbonate aquifer, it is not expected that the increased rate of groundwater withdrawal will significantly affect groundwater flow at the site.

Local:

In the vicinity of the site, there has been little development of the groundwater aquifer because of the poor water quality and the small water requirements of the area users. At the present time, there appear to be no plans for significant development of the groundwater aquifer in the vicinity of the site.

2.4.13.2.3 Piezometric Levels and Groundwater Fluctuations

Regional:

Groundwater in several formations of the carbonate aquifer near the site behaves as though these formations act as a single hydraulic unit. The generalized piezometric surface of this aquifer system in northwestern Ohio is shown in Figure 2.4-25. From Figure 2.4-25, it is evident that, in the area southwest of the site, the piezometric surface has a relatively uniform gradient of approximately 5 ft/mi toward the northeast. In the vicinity of the site, the gradient appears to be less than 5 ft/mi. Water level measurements made in piezometers at the site indicates the groundwater gradient at the site is approximately 2 ft/mi. In general, this gradient is a result of groundwater recharge in the southwest portion of Ottawa County and the northwest portion of Sandusky County. Direction of groundwater movement is from the areas of groundwater recharge northeastward toward Lake Erie.

Regional water level fluctuations in the carbonate aquifer average 3 to 5 ft annually, reflecting influence of normal rainfall conditions. Because the aquifers in the region are generally recharged by vertical seepage through overlying glacial soil deposits, the water levels in the aquifers do not respond immediately to rainfall. In general, there are periods of 2 to 3 days before peak levels in water wells reflect recharge from a heavy rainfall (ref. 32). Daily fluctuations in original water levels are primarily due to changes in atmospheric pressure.

Local:

The site is underlain by glaciolacustrine and till deposits (total thickness of approximately 17 feet) which overlie the Tymochtee formation. Glaciolacustrine and till deposits at the site basically consist of silty clay with very low permeability. The presence of these less permeable soil deposits overlying the Tymochtee formation has created an artesian groundwater condition in water-bearing Tymochtee and underlying carbonate bedrock formations.

Elevation of the piezometric surface of the confined groundwater was determined at the site of rock probes, borings, and piezometers whose locations are shown in Figures 2C.4-1 and 2C.4-2 of Appendix 2C. The approximate piezometric surface in the vicinity of the site before construction was determined in 12 existing water wells.

Elevation of the piezometric surface at the site is generally a few feet above mean water level of Lake Erie; Lake Erie mean water level is approximately 570 feet (I.G.L.D.). The elevation of the lake water level varies slightly with the seasons and with weather conditions. Fluctuations of the piezometric level at the site generally lag behind and are of smaller amplitude than the fluctuations of the lake level.

Piezometric surface elevation also fluctuates with variations in rainfall and atmospheric pressure. The piezometric gradient at the site is small and essentially equal to the gradient of the local rivers and creeks (approximately 2 ft per mile). The gradient is toward Lake Erie. Assuming the carbonate bedrock aquifer to be homogeneous and isotropic with an average permeability of 1x102 cm/sec, an average gradient of 2 ft/mi will result in a calculated average groundwater flow velocity of approximately 5 ft/yr.

Possible Reversal of Groundwater Flow Direction:

In general, groundwater flow is toward Lake Erie. Reversal of groundwater flow direction is not considered likely on a regional basis as a result of the large recharge potential from the southwest, see Figure 2.4-25. However, intermittent reversal of groundwater flow direction on a local basis may occur within a few hundred feet of the Lake Erie shoreline during intermittent increases in lake level that occur as a result of weather conditions. These reversals will have no adverse effect on the safety of the station and are expected to have a total time of occurrence governed by the duration of the lake level increases.

2.4.13.3 Accident Effects

Migration of radioactive contaminants through the soil deposits and bedrock at the site will be affected by groundwater gradients, absorption, and ion-exchange capacities. Gross ion-exchange capacities of the soil deposits and bedrock at the site have not been measured. Because of the complexities from the presence of several different absorbents, different radioactive ions, the presence of interfering chemical constituents in the groundwater, and the nonhomogeneity of the soil, it would be difficult to predict the rate at which the radioactivity of released contaminants would attenuate without elaborate laboratory experiments. However, neglecting the effect of absorption of radioactive materials by the soil deposits and bedrock, the possibility of extensive contamination of the groundwater aquifer (assuming present groundwater conditions remain) is considered low because (1) the redundant safety features incorporated into the construction of the station, and (2) piezometric gradients and corresponding groundwater flow velocities in the site vicinity are small at the present time (1972).

The following is the analysis performed to determine the effect of a ground water radwaste spill on the nearest potable water supply.

<u>Determination of Ground Water Path</u>

The most conservative (shortest) path for ground water movement from a postulated auxiliary building leak to the nearest potable water supply was determined on the basis of the following assumptions:

a. The nearest potable water supply is Lake Erie.

b. Ground water will move downgradient to Lake Erie from the leak location along a straight path,

c. The shortest (and fastest) route of travel is within the most permeable aquifer.

d. Once the ground water underlies the shoreline, it will encounter Lake Erie water, independent of the lake bed conditions.

Based on these assumptions, the most conservative path to Lake Erie from the auxiliary building is to the northeast through the confined dolomite aquifer, a minimum distance of 3,000 feet.

<u>Determination of Ground Water Velocity</u>

The travel time of ground water is dependent on the velocity of the water and the distance traveled. The velocity of the ground water along the most conservative path is calculated using the Darcy equation:

$$V=KI/n$$

where: V = velocity of ground water
K = coefficient of permeability of the aquifer
I = gradient of the potentiometric surface
n = effective porosity of the aquifer

The permeability coefficient K was determined from falling and constant- head pump tests conducted on the dolomite aquifer. The maximum value of K from the field tests was less than 1×10^{-2} cm/sec (10,344 ft/year). However, a conservative value of $K = 1 \times 10^{-2}$ cm/sec was used to determine V.

The gradient I of the confined water in the dolomite aquifer is reported to range between 1 and 2 ft/mi toward Lake Erie. This regional gradient is measured eastward toward Lake Erie from the recharge area west of the plant. Although the shortest ground water path to the lake is toward the northeast, and therefore not along the maximum gradient, the highest value of I (2 ft/mi or 0.0004) was used to determine V.

The velocity of ground water within the aquifer is inversely proportional to the effective porosity n. A conservative value of n was calculated from minimum values of joint and bedding plane separations and maximum values of spacings between joints and bedding planes. These values were determined from boring logs and scaled photographs of the aquifer. The combination of minimum separation values and maximum spacing values gives the lowest (most conservative) value for effective porosity. In addition, intergranular porosity was neglected. Values of n were calculated for a 23-foot-thick section of the aquifer, encompassing three units having slightly different joint and bedding characteristics. Since the aquifer dips

toward Lake Erie at a higher gradient than the potentiometric surface, ground water moving toward the lake from the plant would have to pass through all three of the units. The average n was calculated to be 8.6 percent (0.086) with a range from 5 percent to 16 percent. A conservative value of n = 5 percent (0.05) was used to calculate V.

From the above values, the velocity of ground water from the auxiliary building to Lake Erie through the confined aquifer is 83 ft/year.

Determination of Travel Time

The travel time is calculated from:

T = L/V

where: T = travel time
L = distance traveled
V = velocity of ground water

Using the minimum value of L = 3000 ft and V = 83 ft/year, the travel time of ground water from the auxiliary building to Lake Erie through the confined aquifer is 36 years.

Radionuclide Migration

In the event that a spill from the radwaste tanks would occur, it is assumed that it will enter the groundwater body through a crack or cracks in the building walls. The movement of the radionuclide in the aquifer is controlled by the aforementioned hydraulic properties of the aquifer, type of rock, and the type of radioactive material. Movement of the ion through the media is associated with ion exchange, resulting in a smaller fraction of the ions in the moving water, Grove (1970) and Fenske (1968). The average velocity of the ion traveling through the media is considerably smaller than the water velocity. Hydraulic dispersion in the media is another factor that reduces the ion concentration.

In analyzing the effect of radioactive spill into the ground water at Davis-Besse, a one-dimensional dispersion model with ion exchange is utilized. Longitudinal dispersion is only considered in this study since lateral and vertical dispersion are significantly smaller in magnitude than longitudinal. This assumption will result in a higher concentration at a given point than the actual case.

The one-dimensional dispersion equation with ion exchange through the porous media is expressed as (Grove 1970 and Fenske 1968):

$$\frac{C}{C_0} = \frac{1}{2}\left[\operatorname{erf}\frac{X - U_i t_1}{2\sqrt{D_L \frac{t_1}{R}}} - \operatorname{erf}\frac{X - U_i t}{2\sqrt{D_L \frac{t}{R}}}\right]$$

Where: U_i = ion velocity

$$= \frac{U_W}{1 + d_f \zeta_e} = \frac{U_W}{R}$$

U_W = water velocity through the pore space

d_f = distribution factor - a function of the porous media and the ion

ζ_e = is a coefficient - a function of the bulk density and the porosity

D_L = longitudinal dispersion coefficient

t = time since spill occurred

t_1 = $t - t_0$

t_0 = is the duration of the spill into the porous media.

Longitudinal dispersion coefficient, D_L, was computed using relations developed from laboratory testing on porous media (Harleman & Rumer 1963). A coefficient of 12.6 ft^2/yr was evaluated and used. This value is very conservative since longitudinal dispersion coefficient in rock is significantly higher (Fenske 1968).

The distribution coefficient, d_f, was evaluated from the data presented by Grove (1970) for limestone and dolomite for a given ion. Using the lower limits of the data for Cesium, a coefficient of 50 was selected and for Strontium a value of 8 was selected. A value of 2.5 was used for ζ_e (Grove, 1970).

Utilizing the ground water velocity of 83 ft/yr. the aforementioned distribution coefficients, and without considering decay, it will take about 4,550 years for peak Cesium concentration to reach the lake and about 760 years for Strontium.

In this analysis, the sensitivity of t_o was tested by using 0.5 year and 1 year. The results of the analysis which are presented below shows slight variation in the maximum concentration at the assumed discharge point in the lake.

ION	C/C_0	
	t_0 = 0.5 yr	t_0 = 1 yr
Cesium	5×10^{-3}	1×10^{-2}
Strontium	3×10^{-2}	6×10^{-2}

<u>Radioactive Decay</u>

Assumptions:

1. Primary coolant fission product inventory based on 1 percent failed fuel was used.

2. Credit for radionuclide removal by the plant process systems, consistent with the decontamination factors in WASH-1258 was taken.

3. Failure of the clean waste receiver tank was assumed since they contain the largest inventory of radioactive elements.

Results

The 36-year transit time (T) to Lake Erie should be sufficient time for most of the radionuclides that could be released to decay to insignificant levels. The only exceptions are Sr-90 and Cs-137 which have a half-life of 27.8 years and 30.3 years, respectively. In the case of Strontium and Cesium nuclides their ion exchange properties will produce transit times of 4,550 years for Cesium and 760 years for Strontium as discussed above.

A list of the resulting concentrations at Lake Erie for the radionuclides of greatest concentration (in the clean water receiver tank) and largest half-life is given below. The resulting concentrations for Cesium-137 and Sr-90 are also given.

Isotope	Clean Waste Receiver Tank Conc., μ ci/cc	Half-Life, min	$\lambda(\min^{-1})$	Reduction Factor, $e^{-\lambda T}$	Resulting Conc. at lake, μ ci/cc	10CFR20 Limits, μ cci/cc
Co-58	2.2×10^{-3}	1.03×10^{5}	6.74×10^{-6}	1.2×10^{-57}	2.6×10^{-60}	9×10^{-5}
Rb-88	2.7×10^{-3}	17.7	3.92×10^{-2}	~ 0	~ 0	---------
Mo-99	4.12×10^{-2}	3.98×10^{3}	1.74×10^{-4}	~ 0	~ 0	2×10^{-4}
I-131	3.2×10^{-3}	1.16×10^{4}	5.97×10^{-5}	~ 0	~ 0	3×10^{-7}
I-133	3.8×10^{-3}	1.22×10^{3}	5.69×10^{-4}	~ 0	~ 0	1×10^{-6}
Ba-137m	0.117	2.55	0.272	~ 0	~ 0	----------
[For Cs T = 4,550 years = 2.3×10^{9} min; Sr T = 760 years = 4.0×10^{8} min.]						
Sr-90	1.5×10^{-7}	1.46×10^{7}	4.75×10^{-8}	5.6×10^{-9}	8.4×10^{-16}	3×10^{-7}
Cs-137	0.127	1.59×10^{7}	4.36×10^{-8}	2.8×10^{-44}	3.5×10^{-45}	2×10^{-5}

Conclusion:

Significant delay for all nuclides due to ground water velocity and additional delay for Strontium and Cesium due to ion exchange properties reduces concentrations at the lake to negligible levels. Even if the specific activity chosen to be released was that of primary coolant the concentrations at the shoreline would still be well below 10CFR20 limits. Therefore, there is no effect on the nearest potable water supply.

Also, if a delay time of only 7 years were used for all isotopes except Cs-137 and Sr-90, the reduction at the shoreline would still be significantly below 10CFR20 limits.

References

1. Grove, D. B., "A method to describe the flow of radioactive ions in ground water," USGS and Sandia Laboratories, Dec. 1970.

2. Fenske, P. R., “Prediction of Radio Nuclide in Ground Water,” US AEC, Nevada Operations Office, Dec. 1968.

3. Harleman, D. R. F., and R. R. Rumer, “Longitudinal and Lateral Dispersion in an Isotopic Porous Media,” Jour. of Fluid Mechanics, Part 3, Vol. 16, July 1963.

2.4.13.4 Monitoring of Groundwater Contamination

The potential for significant contamination of groundwater by release of radioactive material into the bedrock aquifer is considered to be low; see Subsection 2.4.13.3. However, if an accidental release of radioactive material should occur, retrieval and monitoring procedures will be implemented.

2.4.13.5 Surficial Soil Permeabilities

Generalized subsurface conditions in the vicinity of the station site area consist of cohesive glacial, organic marsh, and granular lakeshore soil deposits horizontally overlying stratified sedimentary bedrock. Within a few hundred feet of station structures, some Seismic Class II earthen fill and Seismic Class I and Seismic Class II granular fill were placed. Occurrence, description, and physical properties of the soil deposits, fill, and bedrock in the vicinity of the site area is given in Subsection 2.5.1 and in Sections 2C.6.3 and 2C.6.4 of Appendix 2C.

Based on results of field and laboratory permeability tests and on laboratory index property tests, it is estimated that permeability of soil deposits, fill, and bedrock in the station site area are as follows:

Deposit or Material	Estimated Coefficient of Permeability-cm/sec
1. Cohesive Glacial Soil Deposits	
a. Glaciolacustrine deposit	Less than $1x10^{-6}$
b. Till deposit	Less than $1x10^{-6}$
2. Organic Marsh Soil Deposit	Less than $1x10^{-6}$
3. Granular Lakeside Soil Deposit	$1x10^{-2}$
4. Earthen Fill	Less than $1x10^{-6}$
5. Granular Fill	$1x10^{-3}$
6. Bedrock	
a. From El. 560 to El. 535	$1x10^{-2}$ to $1x10^{-3}$
b. From El. 535 to El. 510	$1x10^{-4}$ to $1x10^{-5}$
c. From El. 510 to El. 490	$1x10^{-2}$ to $1x10^{-3}$
d. Below El 490	$1x10^{-4}$ to $1x1^{-5}$

2.4.14 Technical Specification and Emergency Operation Requirements

The station design and arrangement is such that normal operation can continue during any adverse hydrologically related event that could be reasonably expected. The probable maximum surge level conditions of Lake Erie at the station site resulting from a hypothetical wind storm producing this surge would have no effect on station operation.

The minimum level of Lake Erie resulting from a hypothetical wind storm would result in a lake level such that it would uncover the intake through which cooling water supply is drawn into the station. This could result in a need for reduction of station power level to reduce the amount of

evaporative loss from the cooling tower, but would have no adverse effect on the safety aspects of the station because of the impoundment of water in the intake canal forebay for essential cooling water supply. Any extreme low lake level of this magnitude would be of very short-time duration and would be preceded by weather conditions over a duration of time that would give a minimum of 12 hours advance warning of the probability of this low water preceded by a period of days advance warning of the storm probability.

No need for Technical Specification or Emergency Procedure consideration of these maximum probable lake level conditions is considered to be necessary.

TABLE 2.4-1

Storm Waves and Water Levels

	November 14, 1972			April 9, 1973	
Time	0400	1000	1500	1600	1300
Wind Speed, fps	60.4	31.4	33.8	24.2	36.2
Bearing	N50ºE	N30ºE	N60ºE	N70ºE	N90ºE
Effective Fetch, ft.	122,000	118,100	107,000	92,300	79,000
Wind Setup, ft.	6.8	7.4	7.8	8.1	7.2
Avg Depth, ft.	31.8	35.4	30.0	31.1	25.2
Significant Wave, ft.	5.6	3.0	3.1	2.1	2.9
Significant Period, sec.	3.9	3.1	3.1	2.6	2.9
Maximum Wave, ft.	10.5	5.6	5.8	3.9	5.4
Maximum Period, sec.	4.9	3.9	3.9	3.3	3.6
Supportable Wave, ft.	4.2	4.7	5.0	5.2	4.5
Wave Run-up, ft.		3.7		3.2	
Maximum water elevation, ft. I.G.L.D.		579.7		579.9	

TABLE 2.4-2

Stream Flow Data for Toussaint Creek

Date of Spot Readings	Spot Readings at Limestone, OH	Estimated Total Stream Flow
9/12/67	0.86 C.F.S.*	2 C.F.S.
7/6/65	3.35 C.F.S.*	7 C.F.S.
9/10/64	0.48 C.F.S.**	1 C.F.S.
8/30/63	0.42 C.F.S.**	1 C.F.S.
8/16/62	1.43 C.F.S.**	3 C.F.S.
9/29/61	2.88 C.F.S.**	6 C.F.S.

Sources:

* U.S. Department of Interior Geological Survey, “Water Resources Data for Ohio, Part 1,” 1965 and 1967.

**U.S. Department of Interior Geological Survey, “Surface Water Records for Ohio,” 1961, 1962, 1963 and 1964.

TABLE 2.4-3

Selected 24-Hour Probable Maximum Precipitation

Time (Hours)	Incremental Rainfall (Inches)	Accumulative Rainfall (Inches)
0 - 6	1.00	1.00
6 - 12	2.17	3.17
12 - 15	3.88	7.05
15 - 18	15.50	22.55
18 - 24	1.50	24.05

Note: This table is derived in Reference 61.

TABLE 2.4-4

Estimated Precipitation Losses and Runoff

Duration (hours)	Incremental Precipitation (in)	Incremental Losses (in)*	Incremental Runoff (in)	Accumulative Runoff (in)
0	0	0	0	0.000
3	0.4	0.105	0.295	0.295
6	0.6	0.105	0.495	0.790
9	0.8	0.105	0.695	1.485
12	1.37	0.105	1.265	2.750
15	3.88	0.105	3.775	6.525
18	15.5	0.105	15.395	21.920
21	1.25	0.105	1.145	23.065
24	0.25	0.105	0.145	23.210

Note: This table is derived in Reference 61.

TABLE 2.4-5

Summary of Calculated Peak Setup Elevations Using Platzman's Model and Measured Peak Setup at Toledo, Ohio for Four Storms

Storm Date	Maximum Lake Level feet (I.G.L.D.) Calculated	Measured	Peak Wind Setup (in feet) Calculated	Measured
Apr 26-28, 1966	575.3**	575.6*	5.0**	5.3*
	574.0	573.1	3.7	2.8
Nov 13-15, 1972	576.7	576.0	4.6	3.9
Mar 16-18, 1973	575.5	574.8	2.6	1.9
Apr 7-10, 1973	575.8	574.9	2.4	1.5
	***	576.7*	***	3.3*

Note: Unless otherwise indicated, setup refers to longitudinal components only.

* Includes cross-lake component
** Includes correction for cross-lake component
*** Insufficient wind data to calculate cross-lake component

TABLE 2.4-6

Results of Chemical Analyses of Groundwater and Surface Water Samples

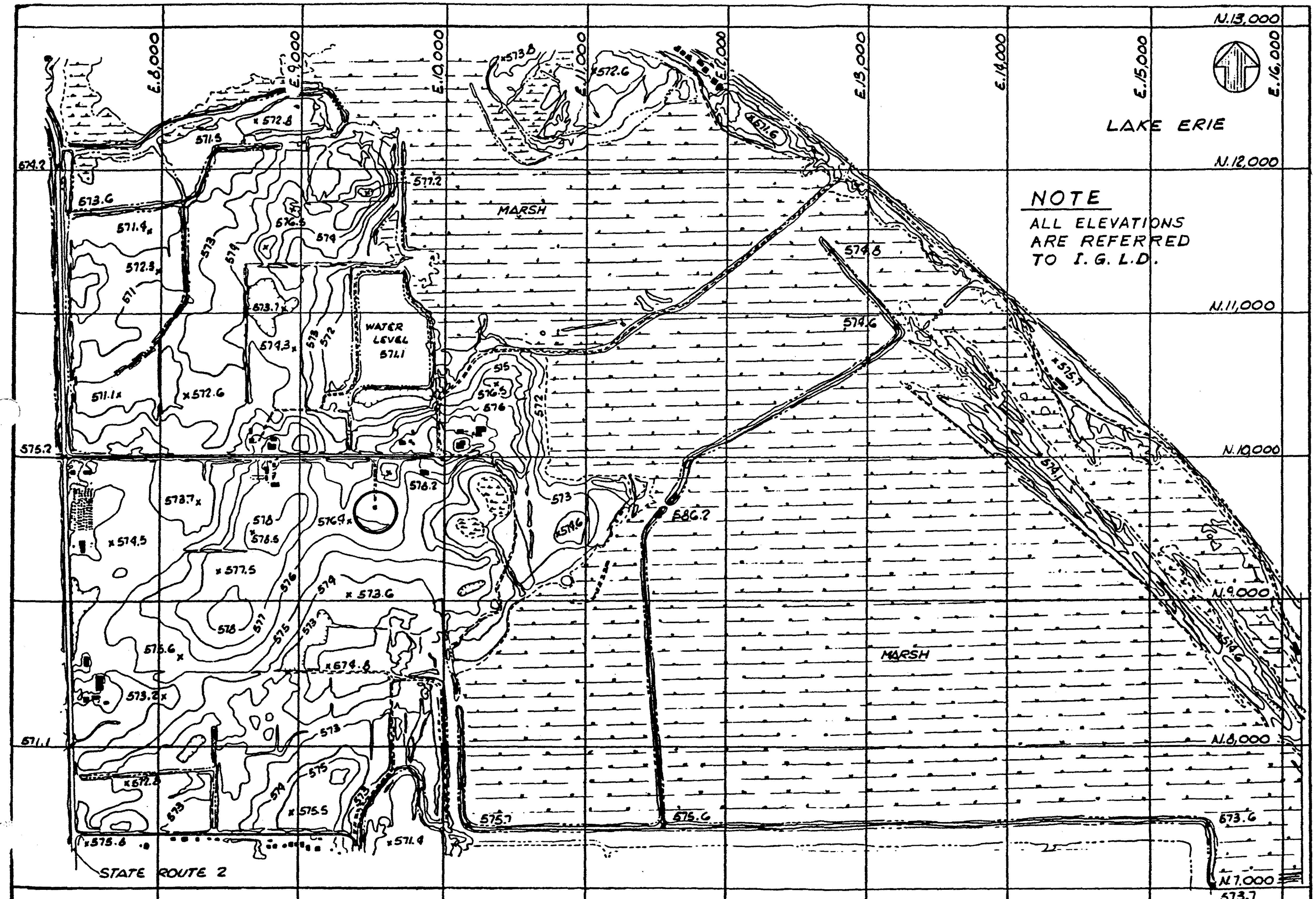
Sample No.	Origin	pH	$Si0_2$	Fe	Mn	Ca	Mg	Total Hard-ness	Na	K	$HC0_3$	$C0_3$	Total Alka-linity	$S0_4$	Cl	F	$N0_3$	TDS
1-1	B2-5 el 464-508	8.0	30.5	<0.1			1800	4500						1700	160			8850
1-1a	B2-5 el 464-508		26	300											2870			8228$^{(*)}$
1-2	B2-5 el 508-520		34.5	0.2			1400	4100						1700	160			8800
1-4	B2-5 el 543-554	8.0	18	3											286			2567$^{(*)}$
1-4a	B2-5 el 543-554	8.0	20	4											290			3719$^{(*)}$
2-9	RP-10 el 560		17.5	1.9	<0.1	465	155	1798	62	5	303	0.0	248	1320	122	1.0	1.0	2301
2-2	test exc Sect 1							1770						1500				3970
2-3	test exc Sect 2							1670						1000				3631
2-8	test exc Sect 2	6.9						2500					268	1300				4509
2-1	test exc Sect 1							1690						1700				3510

TABLE 2.4-6 (Continued)

Results of Chemical Analyses of Groundwater and Surface Water Samples

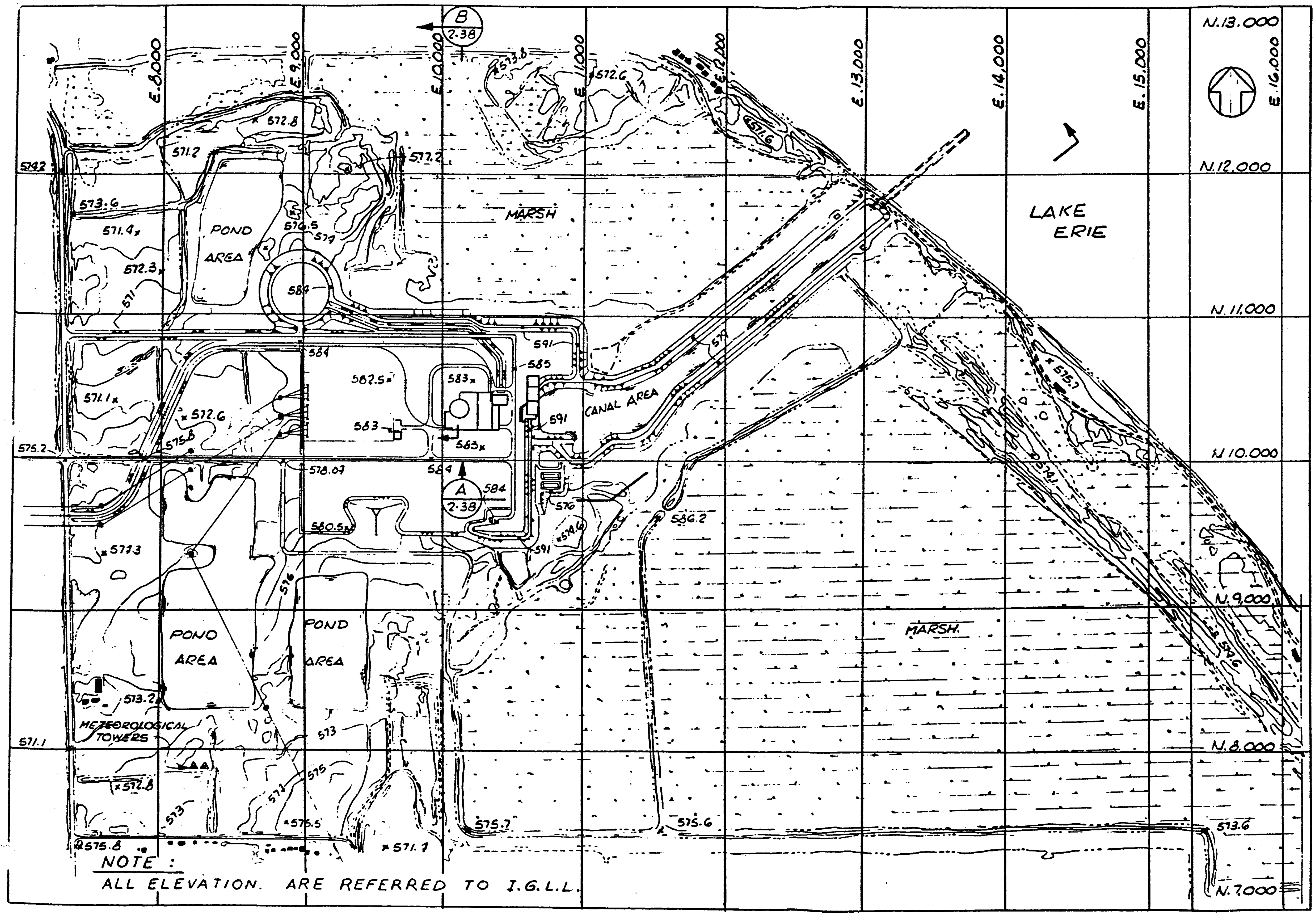
Sample No.	Origin	pH	$Si0_2$	Fe	Mn	Ca	Mg	Total Hard-ness	Na	K	$HC0_3$	$C0_3$	Total Alka-linity	$S0_4$	Cl	F	$N0_3$	TDS
2-11	test exc Sect 2	7.3						2500					282	1400				3309
2-14	farm well G. Finkin	7.2	16.5	1.4	0.04	319	218	1698	41	7.4	354	0.0	290	1340	100	0.8	0.5	2224
2-16	farm well W. Gyde	7.1	12.5	0.4	<0.01	393	197	1790	40	18	192	0.0	162	1550	20	1.2	1.0	2329
2-19	farm well J. Laubocker	7.5	13.0	0.5	<0.01	305	163	1430	68	36	107	0.0	92	1500	30	2.0	0.6	2112
2-21	farm well E. Laubocker	7.4	14.5	0.9	0.01	89	205	1065	38	3.8	305	0.0	254	810	40	0.4	7.6	1361
								Surface Water										
2-4	Toussaint River	7.5	1.0	5.2	<0.01	17.7	54	266	14	2.4	176	0.0	144	85	36	0.2	4.0	307
2-7	Lake Erie	7.5	<0.1	1.3	<0.01	30.5	17.5	148	10	1.6	102	0.0	84	20	20	<0.01	1.5	154

Notes: (a) Except for Sample No, origin, and pH, all numbers are in parts per million by weight
(b) (*) includes dissolved and suspended solids
(c) Sample locations shown in Figure 2.1-8



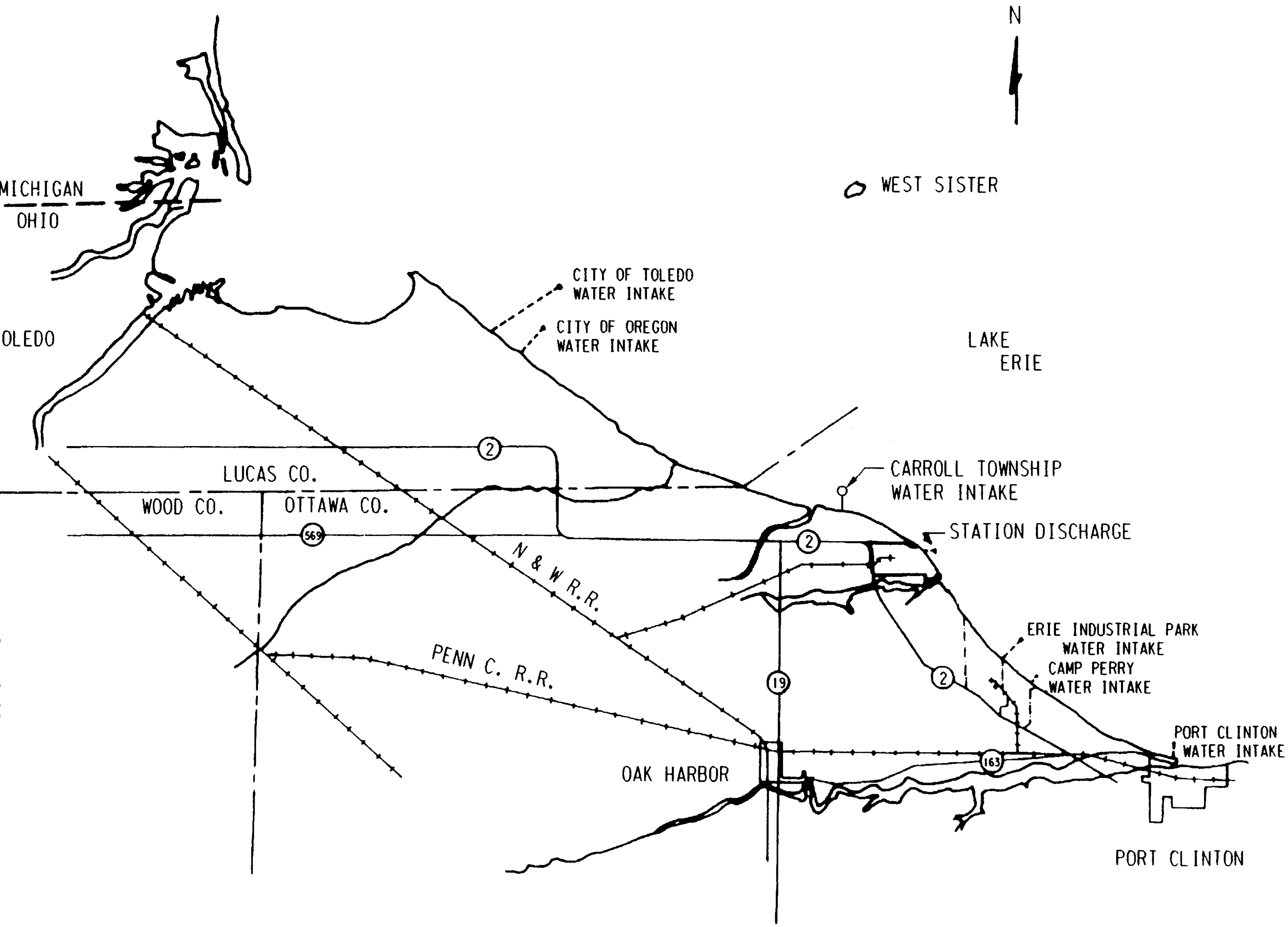
DAVIS-BESSE NUCLEAR POWER STATION
ORIGINAL SITE TOPOGRAPHY
FIGURE 2.4-1

REVISION 0
JULY 1982



DAVIS-BESSE NUCLEAR POWER STATION
FINISHED SITE TOPOGRAPHY
FIGURE 2.4-2

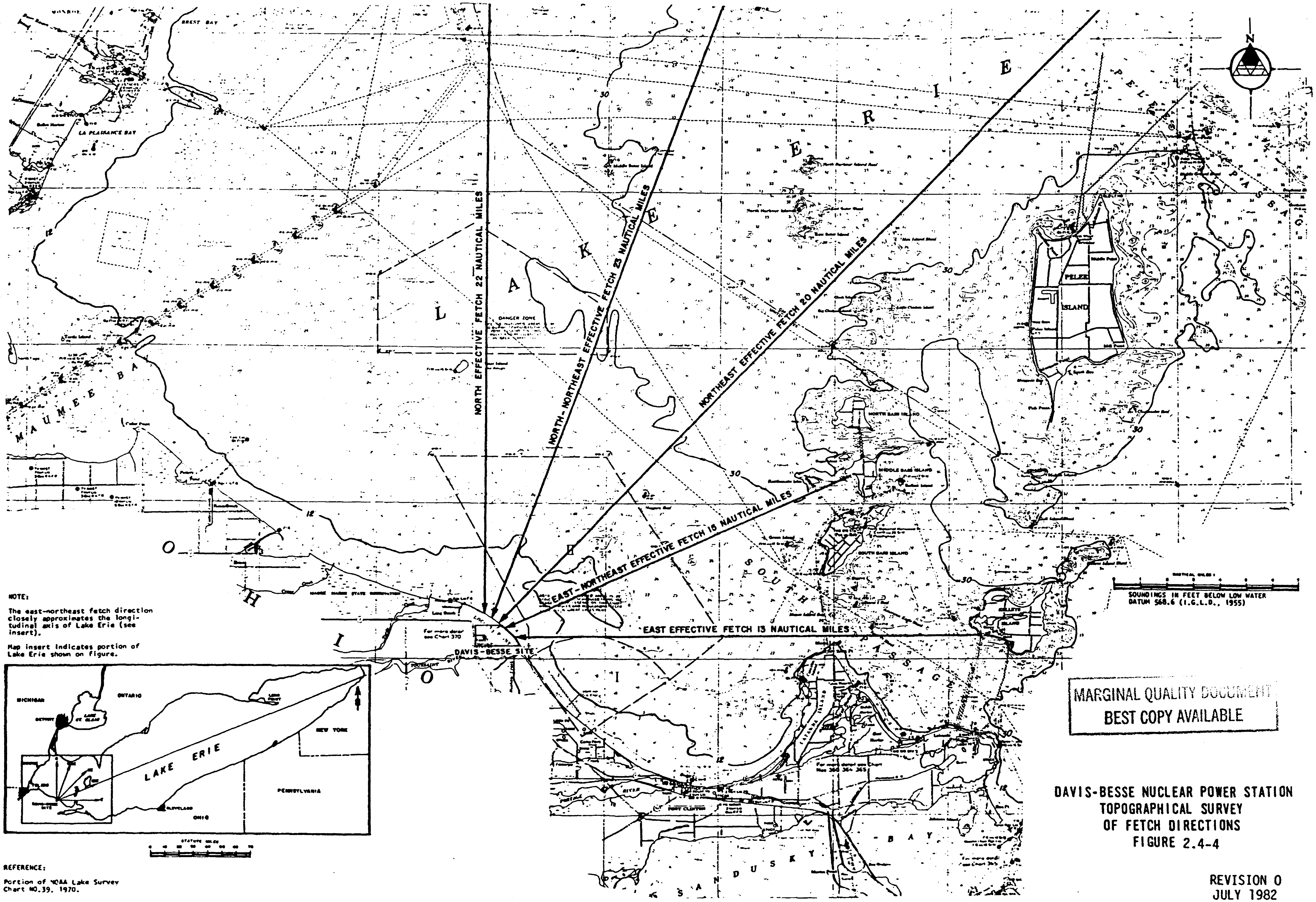
REVISION 0
JULY 1982



DAVIS-BESSE NUCLEAR POWER STATION
WATER INTAKES
FIGURE 2.4-3
REVISION 22
NOVEMBER 2000

DFN: G/USAR/UFIG243.DGN/CIT

DB 07-11-00



NOTE:

The east-northeast fetch direction closely approximates the longitudinal axis of Lake Erie (see insert).

Map insert indicates portion of Lake Erie shown on figure.

REFERENCE:

Portion of NOAA Lake Survey Chart NO.39, 1970.

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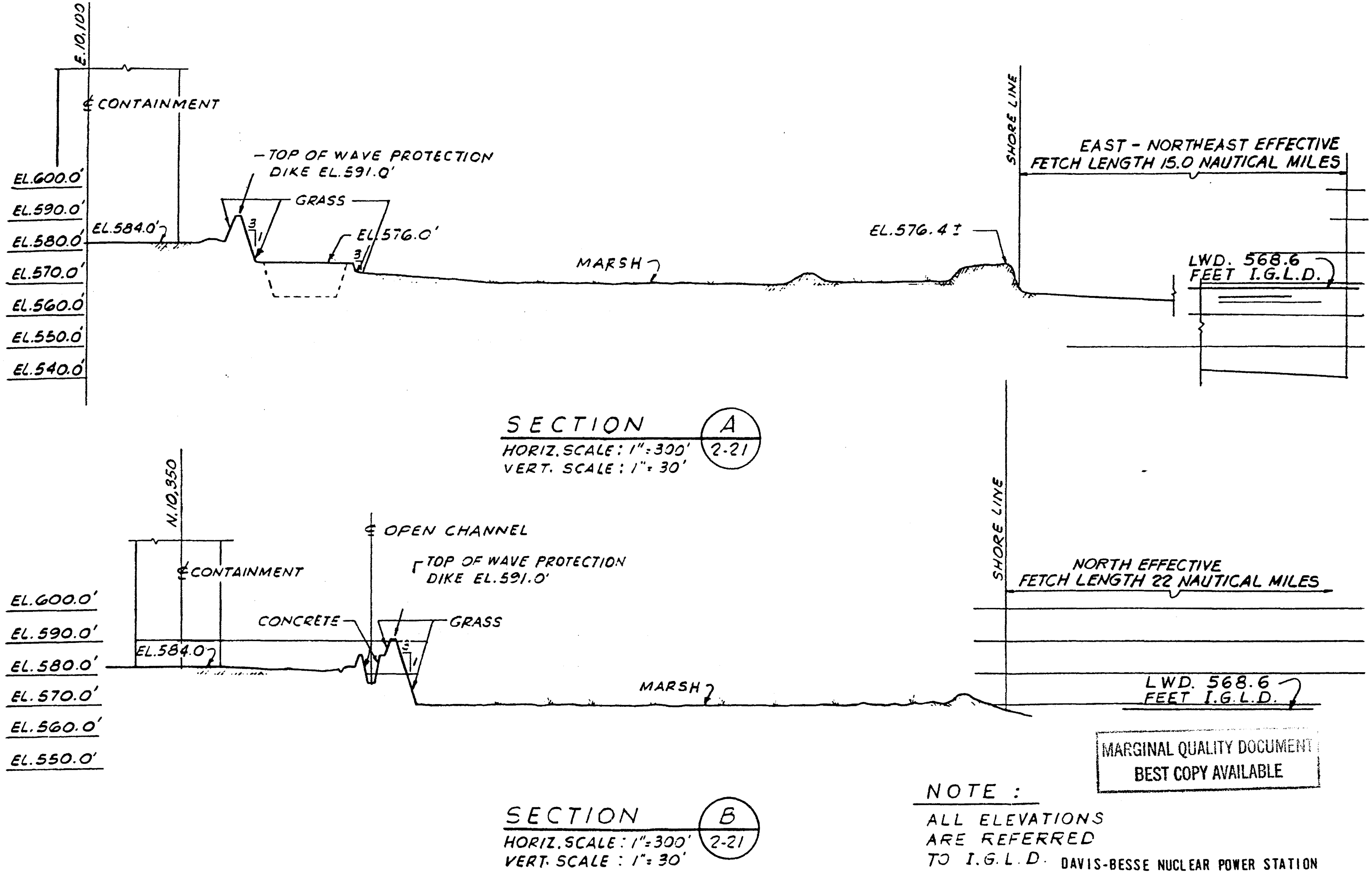
DAVIS-BESSE NUCLEAR POWER STATION
TOPOGRAPHICAL SURVEY
OF FETCH DIRECTIONS
FIGURE 2.4-4

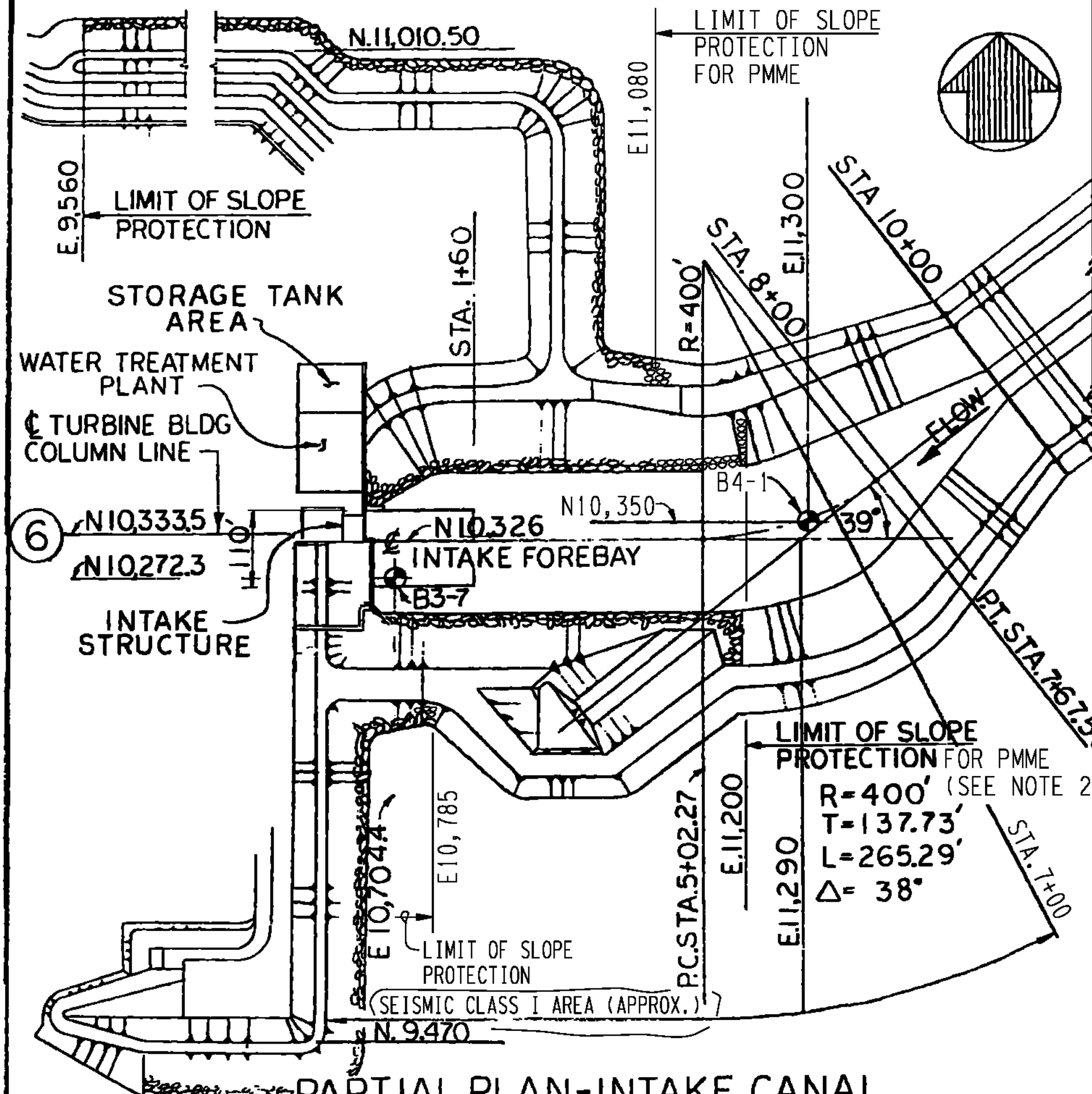
REVISION 0
JULY 1982



NOTE:
ALL ELEVATIONS
ARE REFERRED
TO I.G.L.D.

DAVIS-BESSE NUCLEAR POWER STATION
SECTIONS
FIGURE 2.4-5

REVISION 0
JULY 1982



PARTIAL PLAN-INTAKE CANAL

E.10,350
LIMIT OF SLOPE
PROTECTION

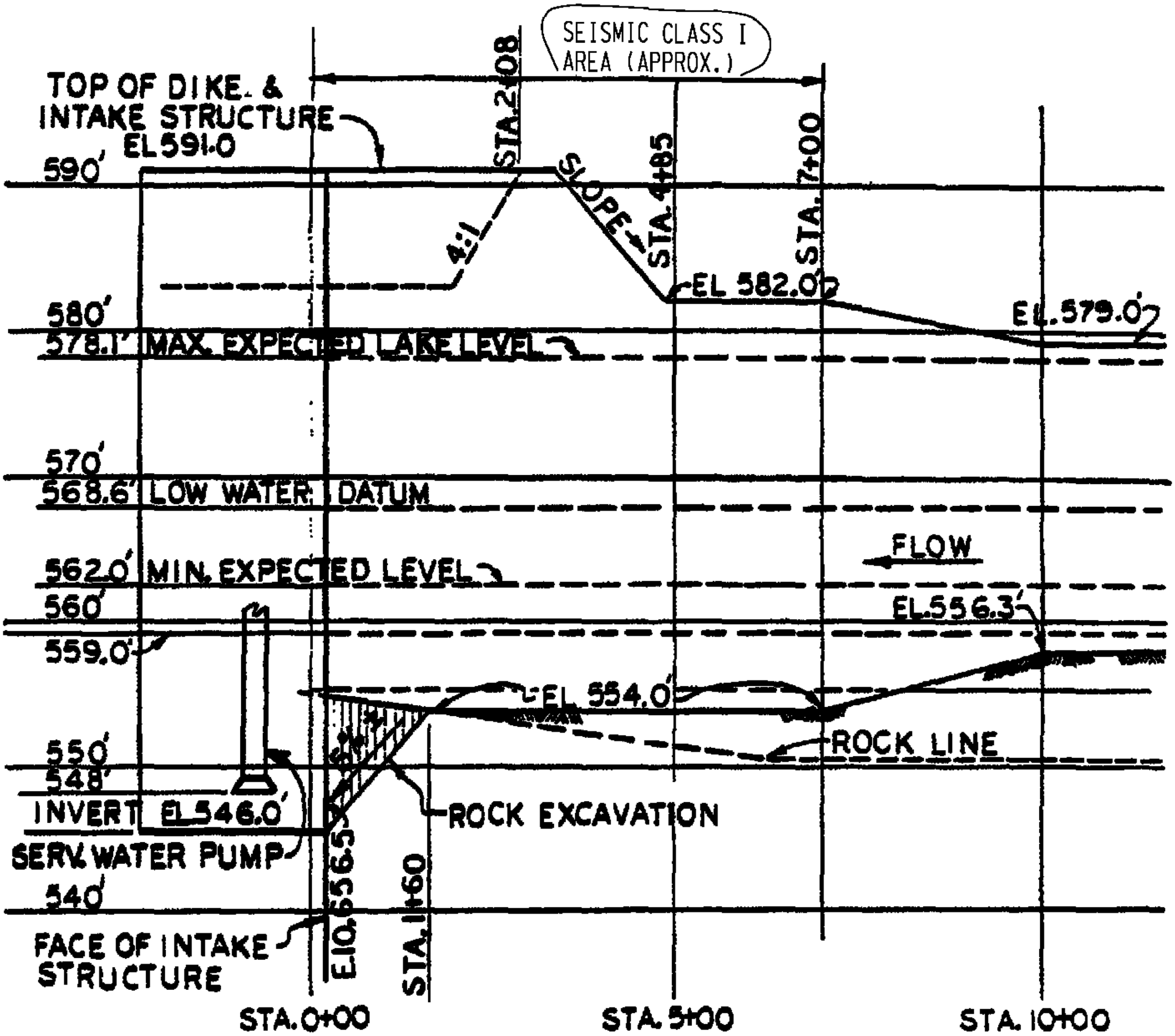
NOTES:

1. ALL ELEVATIONS ARE REFERRED TO I.G.L.D.
2. THE ENTIRE INBOARD SIDES OF THE INTAKE CANAL DIKE ARE LINED WITH RIPRAP.

DAVIS-BESSE NUCLEAR POWER STATION
PARTIAL PLAN OF INTAKE CANAL
FIGURE 2.4-6

REVISION 29
DECEMBER 2012

DB 08-28-12 DFN G:/USAR/UFIG246.DGN/CIT



WATER EL.	VOL. (GALS)	SURF.AREA(S.F.)
568.6'	19,780,000	216,900
562.0'	10,119,000	175,400
560.0'	7,559,000	167,000
559.0'	6,325,500	162,800
557.0'	3,954,000	154,400
555.0'	1,692,000	141,800
554.0'	631,300	19,400
553.0'	494,800	17,200
548.0'	59,100	6,200

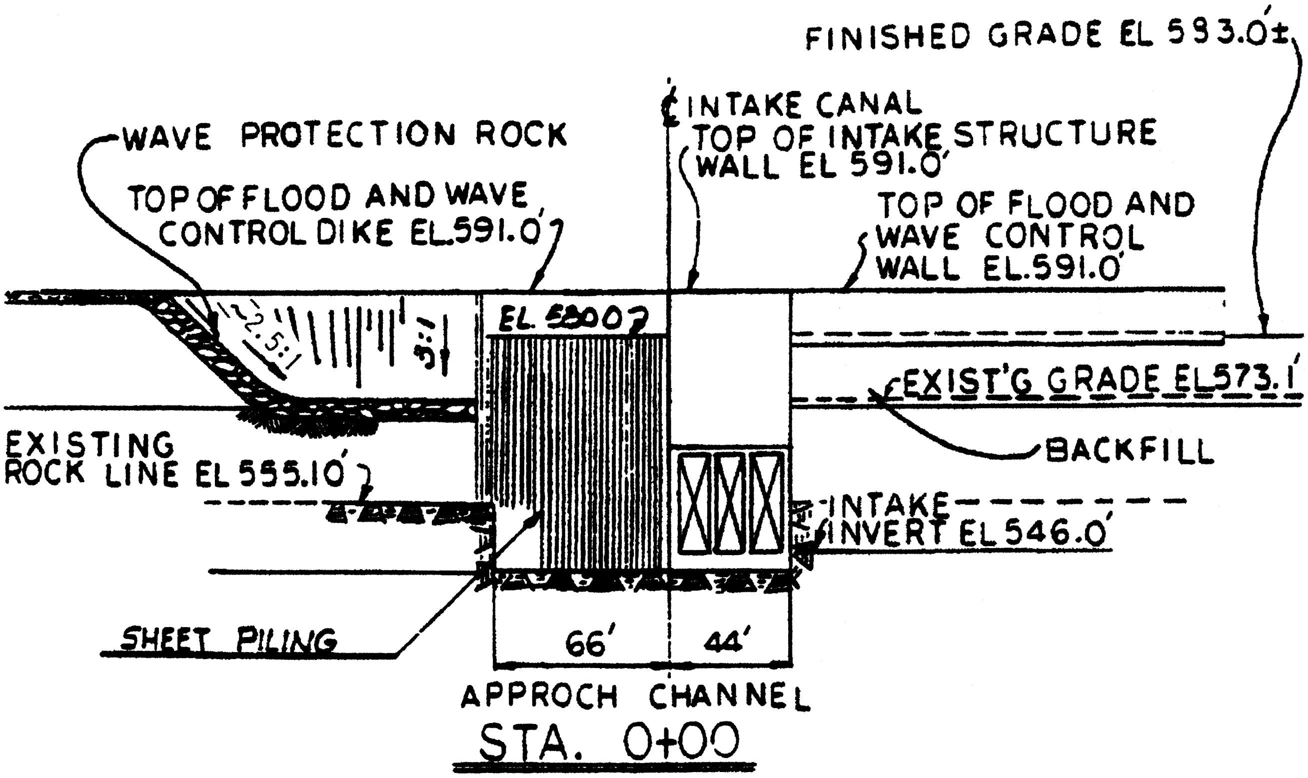
NOTE:

ALL ELEVATIONS ARE REFERRED TO I.G.L.D.

DAVIS-BESSE NUCLEAR POWER STATION
PROFILE ALONG CENTER
LINE OF THE INTAKE CANAL
FIGURE 2.4-7

REVISION 29
DECEMBER 2012

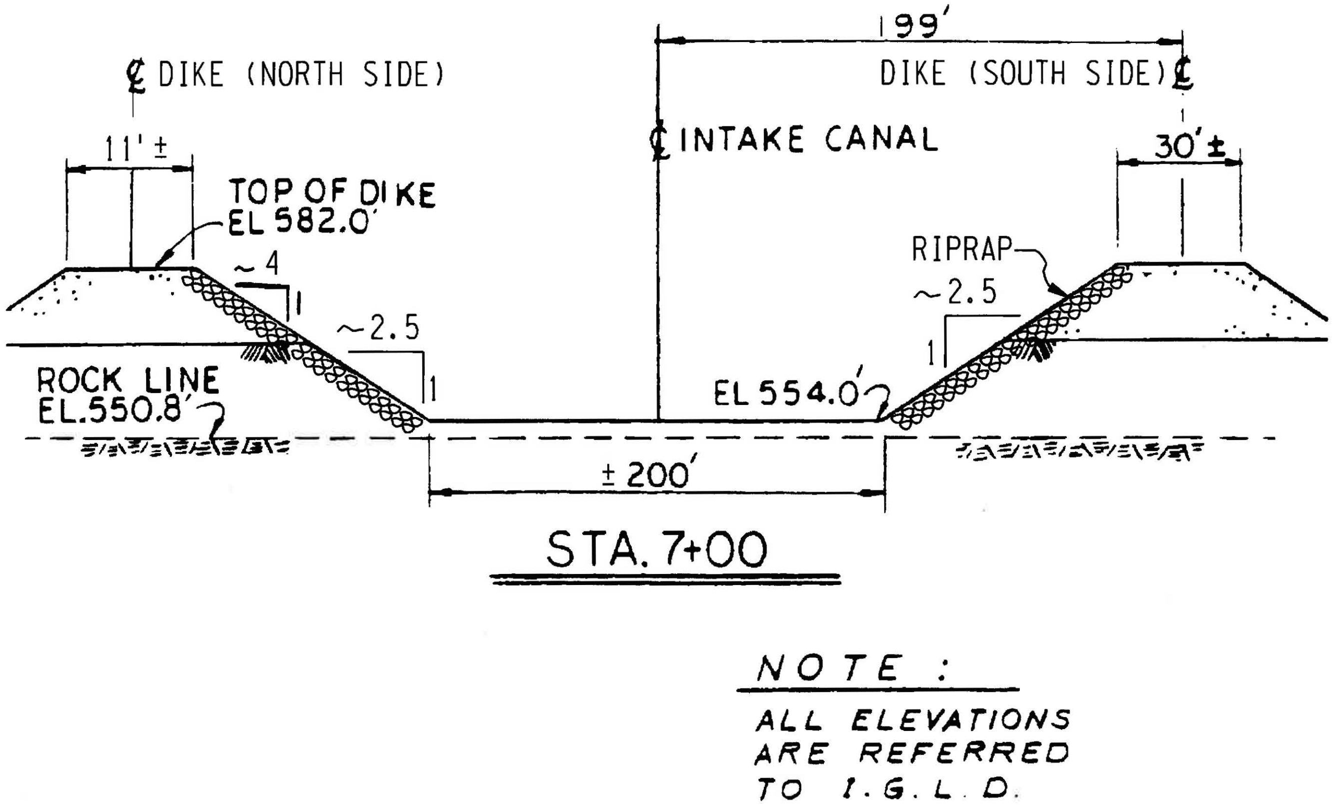
DB 08-28-12 DFN G:/USAR/UFIG247.DGN/TIF



DAVIS-BESSE NUCLEAR POWER STATION
CROSS SECTION @ STA. 0+00
INTAKE CANAL
FIGURE 2.4-8

REVISION 30
OCTOBER 2014

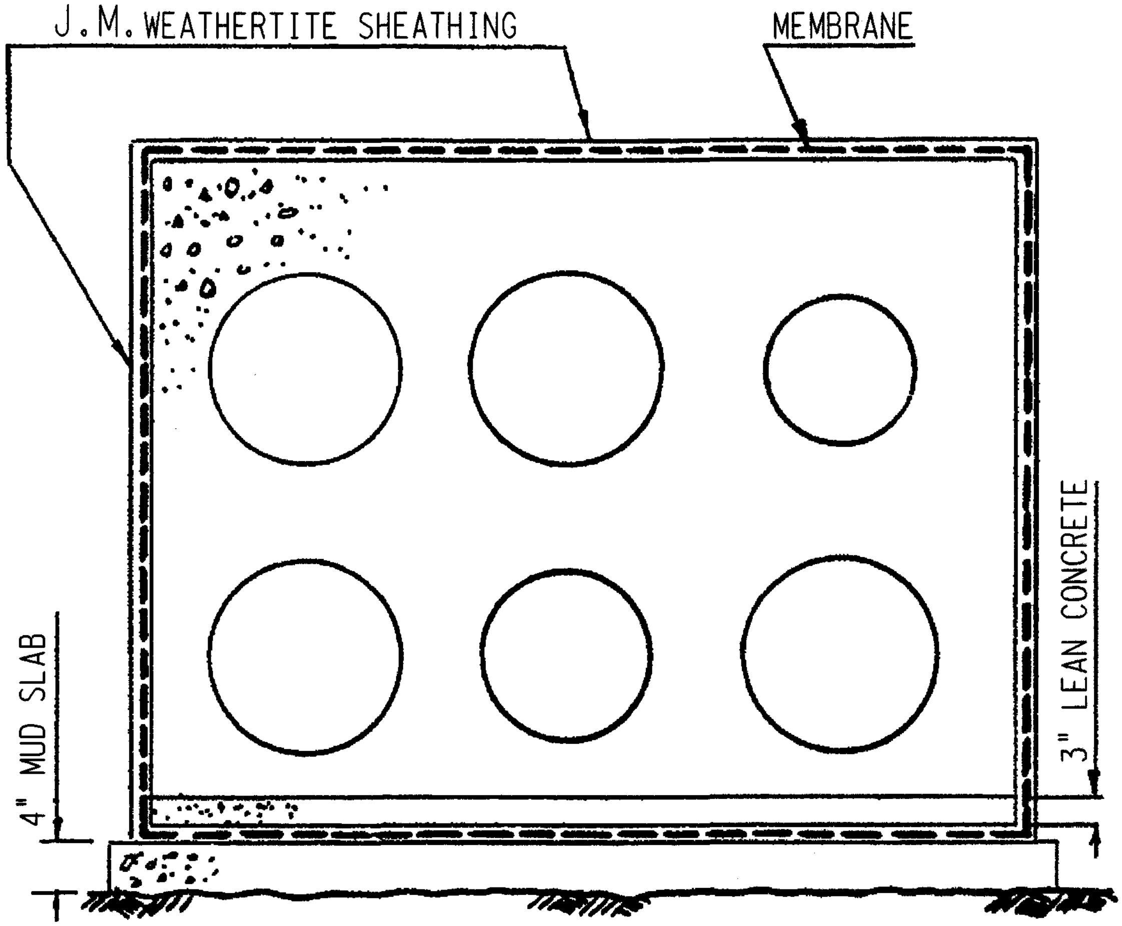
DB 02-17-14 DFN G:/USAR/UFIG248.DGN/TIF



DAVIS-BESSE NUCLEAR POWER STATION
CROSS SECTION @ STA. 7+ 00
INTAKE CANAL
FIGURE 2.4-9

REVISION 31
OCTOBER 2016

DB 03-15-16 DFN G:/USAR/UFIG249.DGN/CIT



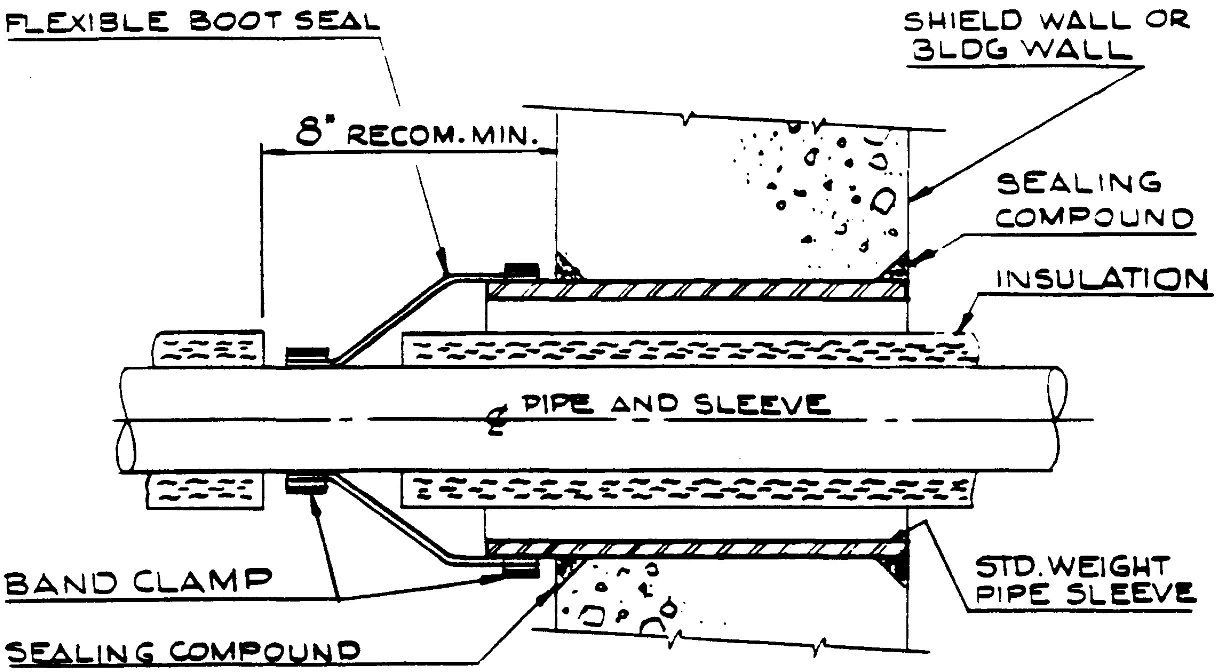
DAVIS-BESSE NUCLEAR POWER STATION

DUCT BANKS

FIGURE 2.4-10

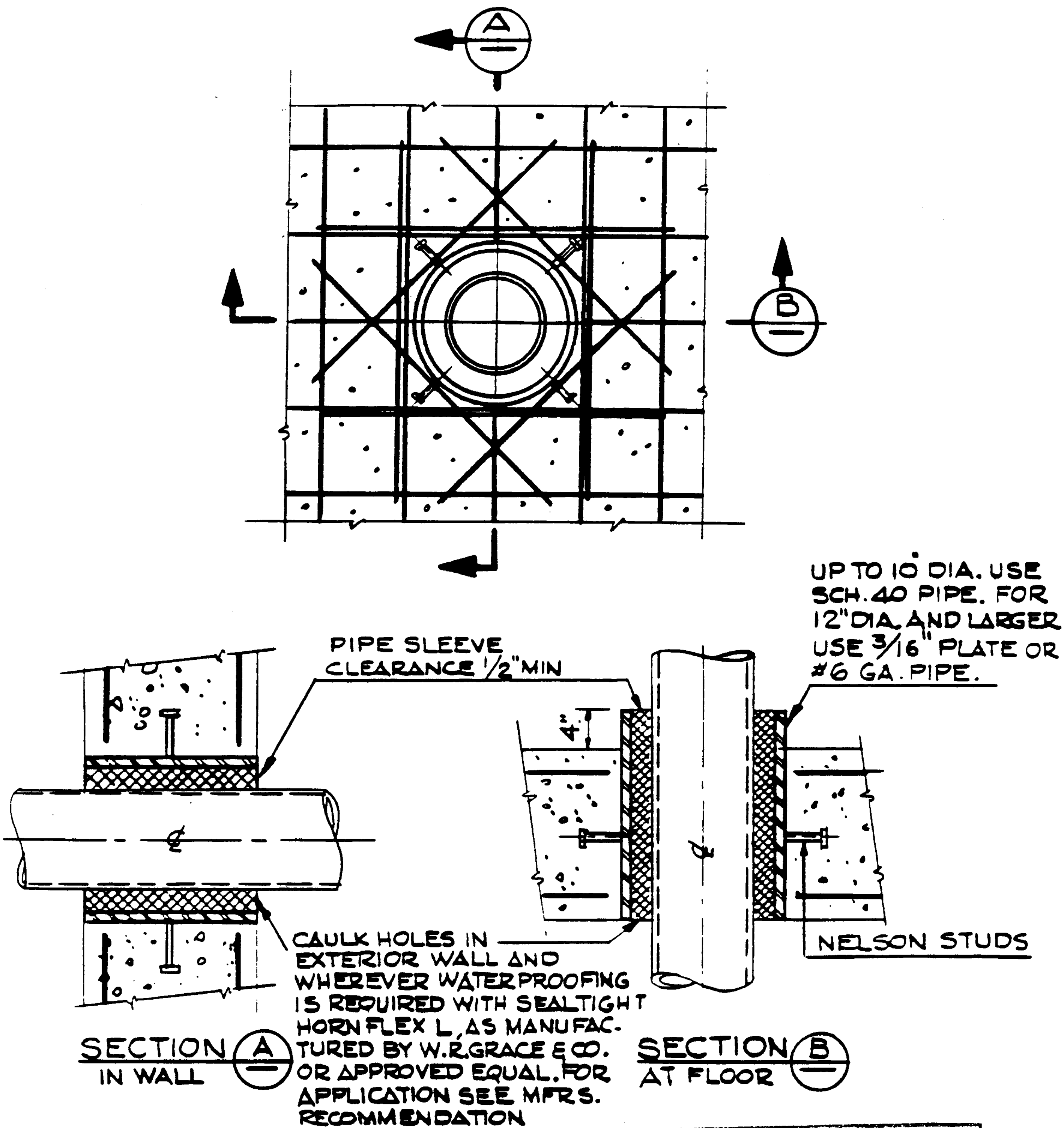
REVISION 26
JUNE 2008

03-07-08 DFN=/G/USAR/UFIG2410.DGN/TIF



DAVIS-BESSE NUCLEAR POWER STATION
BOOT SEAL DETAIL FOR
NEG. PRESSURE & FLOOD
FIGURE 2.4-11

REVISION 0
JULY 1982



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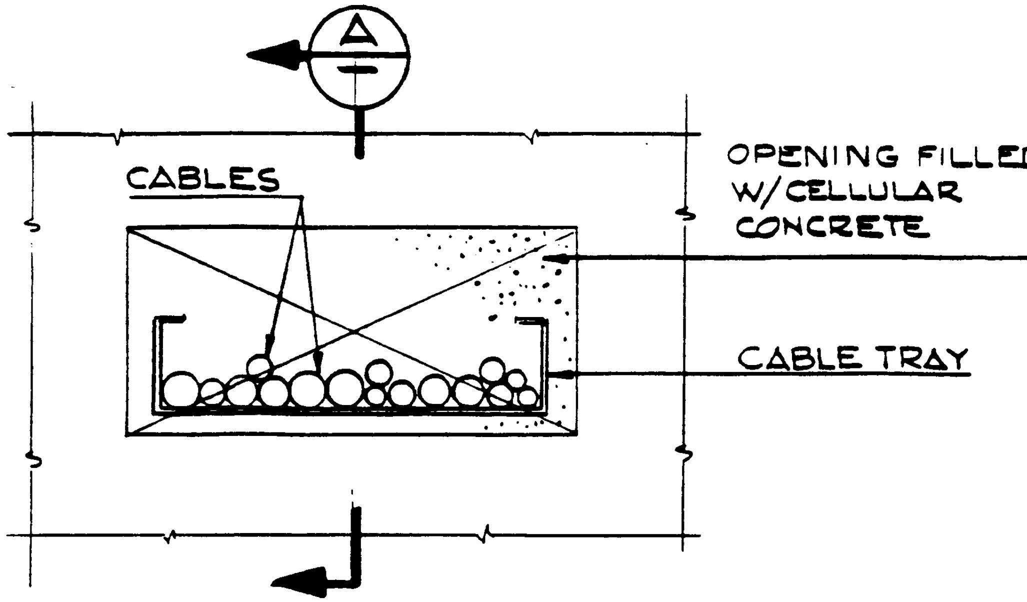
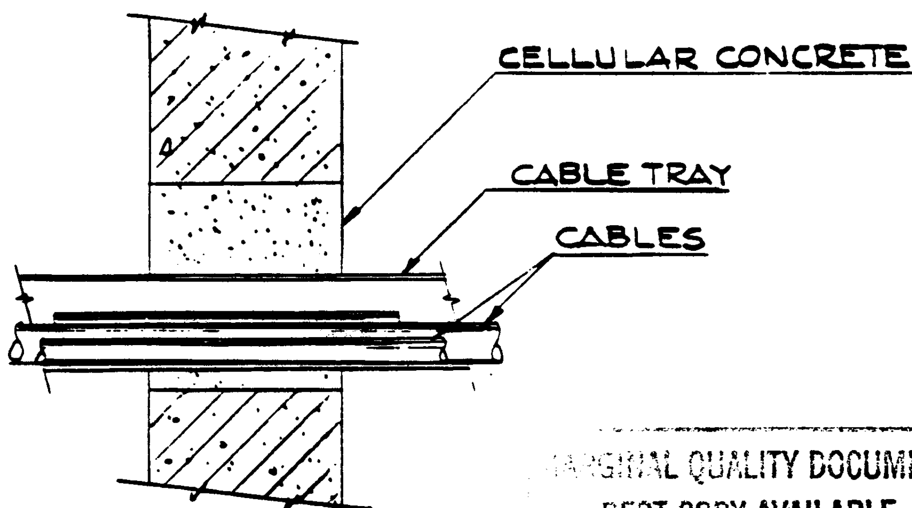
DAVIS-BESSE NUCLEAR POWER STATION
TYPICAL FLOOD PROTECTION DETAIL IN WALLS AND FLOORS
FIGURE 2.4-12

REVISION 0
JULY 1982



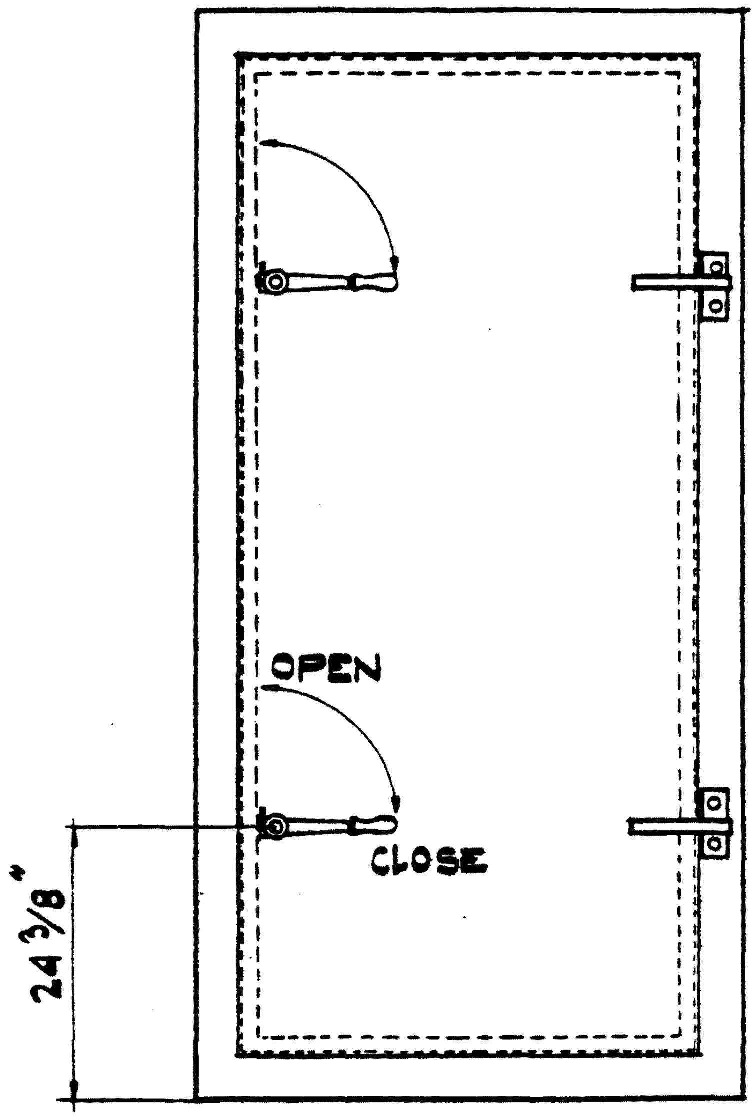
ELEVATION
N.T.S.



SECTION A
N.T.S.

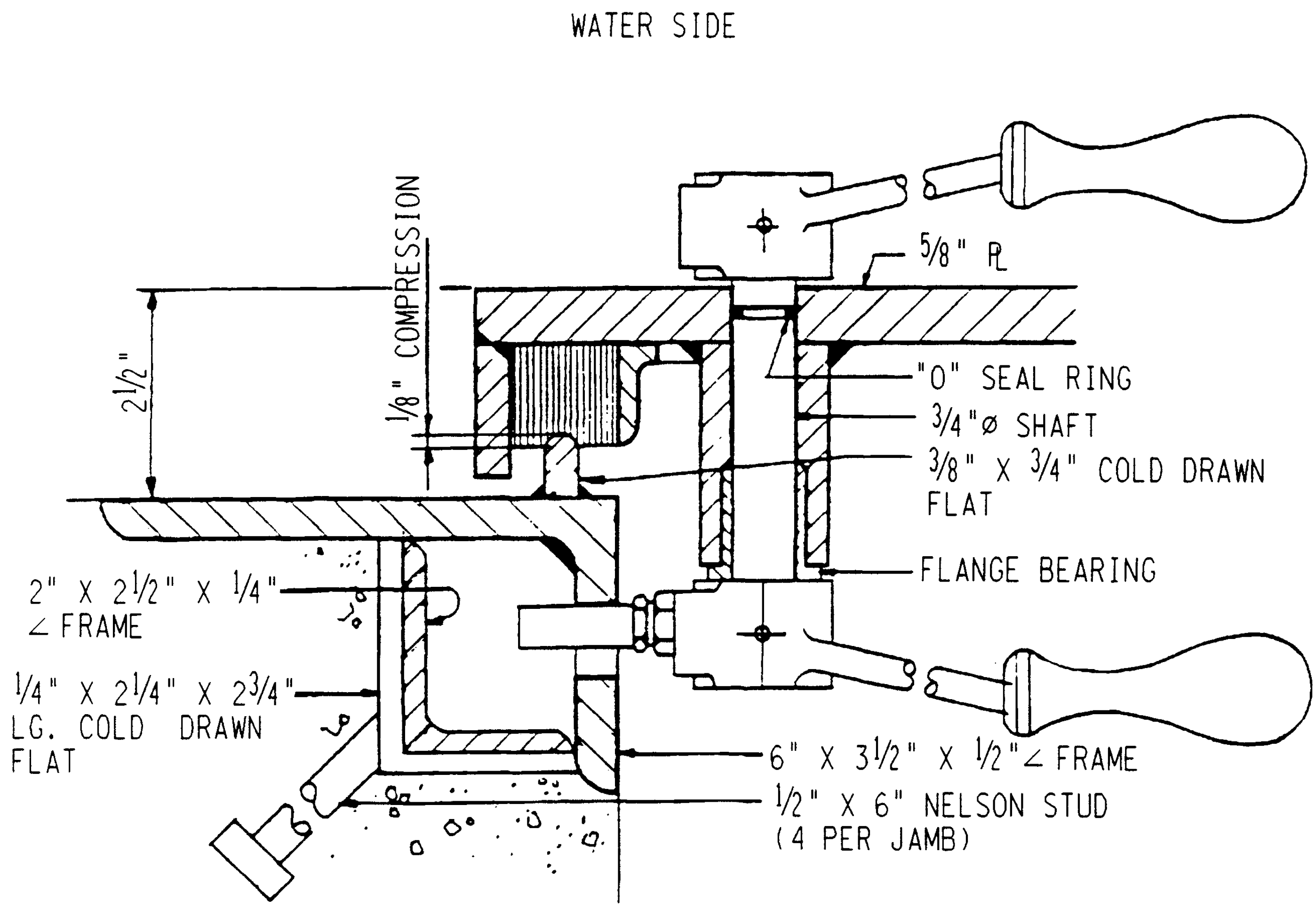
DAVIS-BESSE NUCLEAR POWER STATION
TYPICAL FLOOD PROTECTION DETAIL FOR CABLE TRAYS
FIGURE 2.4-13

REVISION 0
JULY 1982



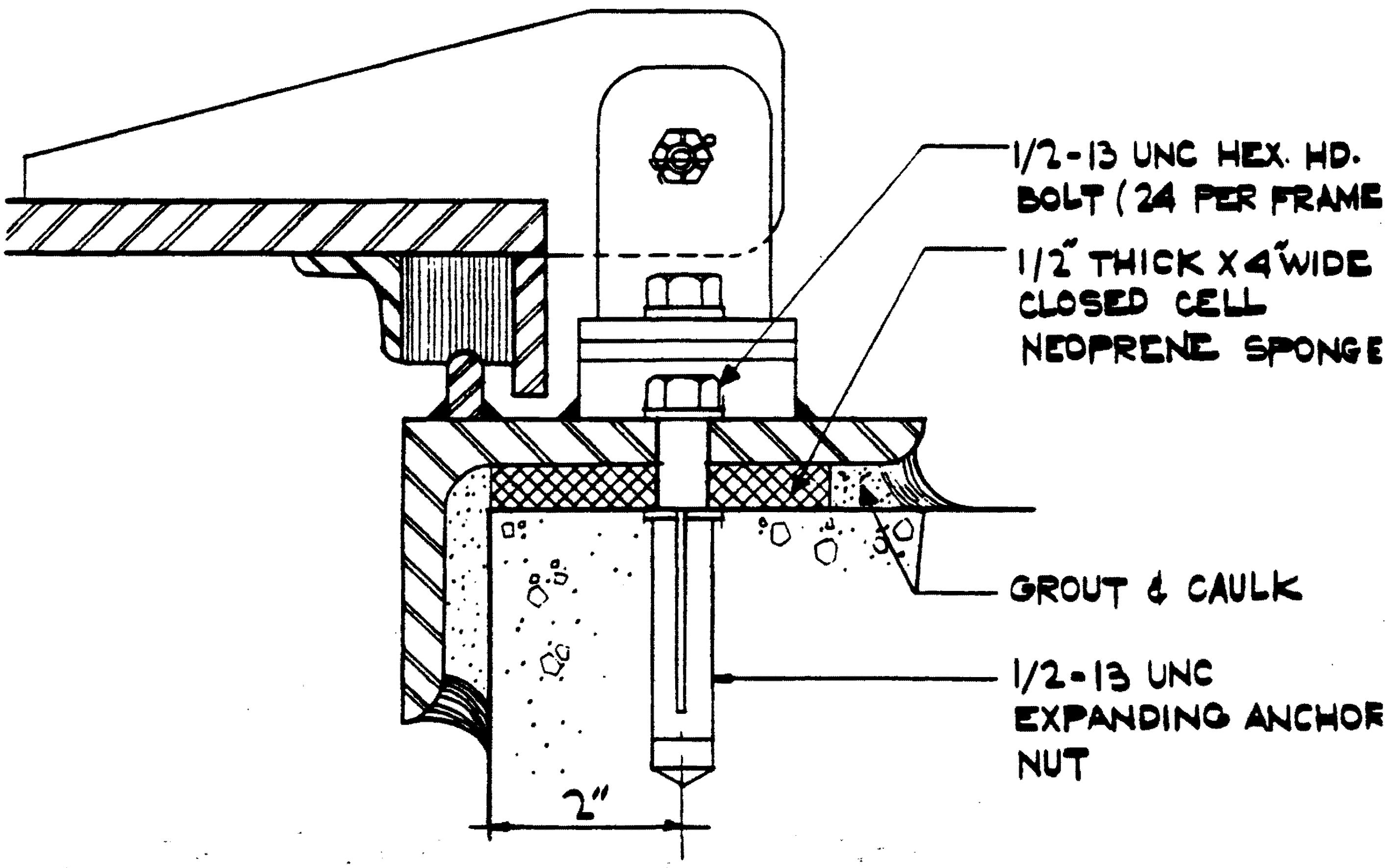
DAVIS-BESSE NUCLEAR POWER STATION
TYPICAL WATERTIGHT DOOR ELEVATION (3'-0'' X 7'-0'' MAX.)
FIGURE 2.4-14

REVISION 0
JULY 1982



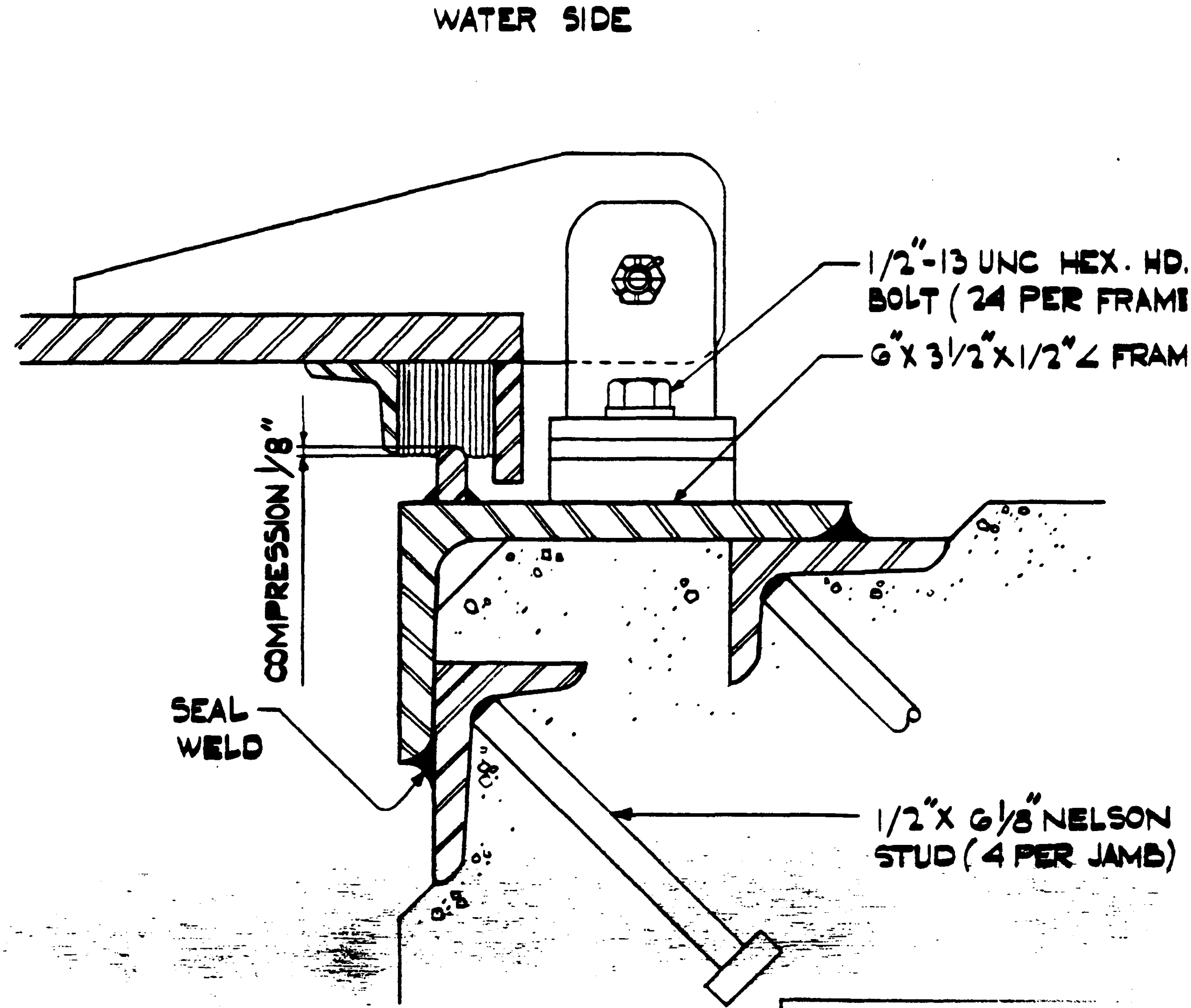
DAVIS-BESSE NUCLEAR POWER STATION
TYPICAL WATERTIGHT DOOR-LATCH JAMB DETAIL
STANDARD INSTALLATION
FIGURE 2.4-15
REVISION 22
NOVEMBER 2000

WATER SIDE



DAVIS-BESSE NUCLEAR POWER STATION
TYPICAL WATERTIGHT DOOR-
HINGE JAMB DETAIL
EXISTING OPENING INSTALLATION
FIGURE 2.4-16

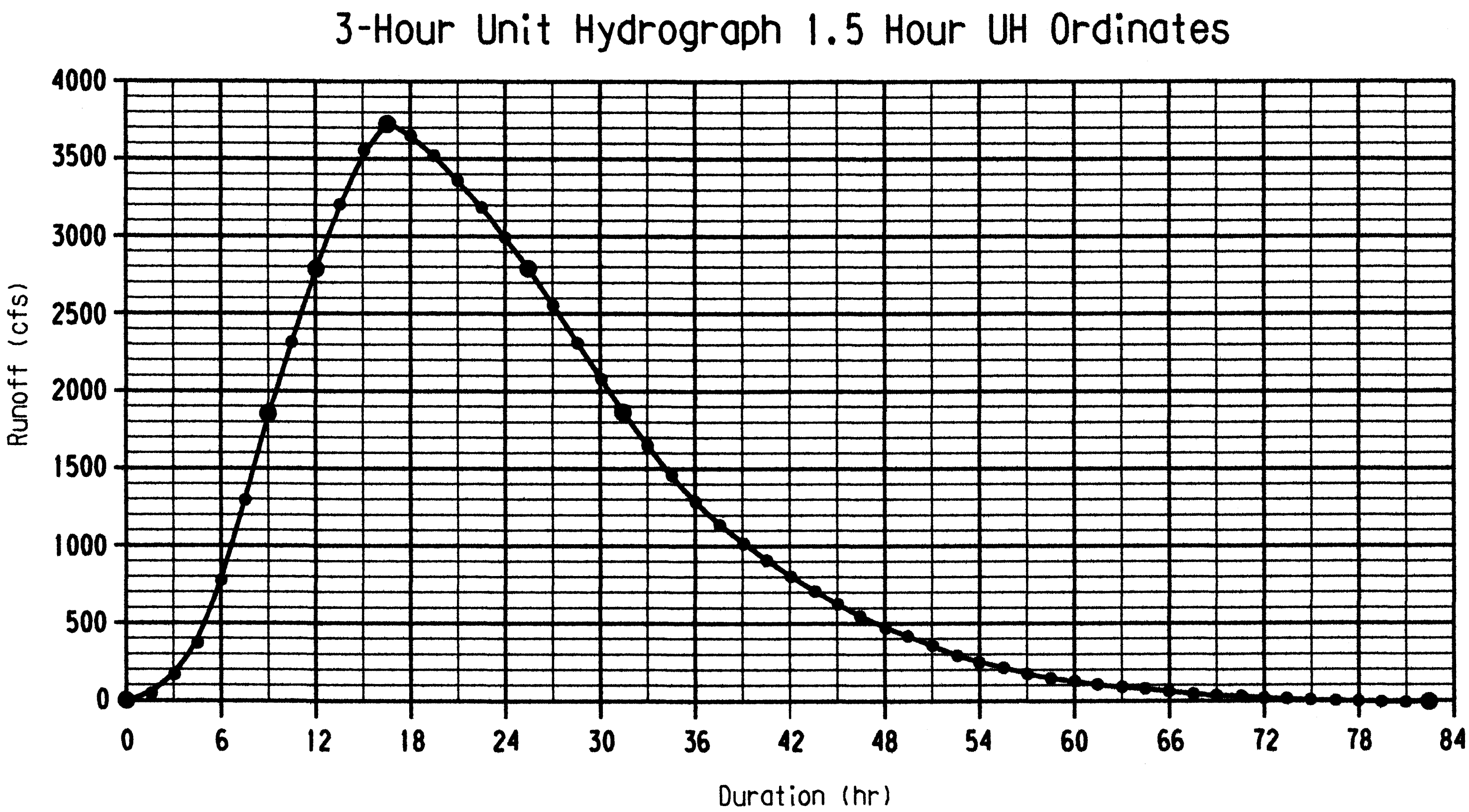
REVISION 0
JULY 1982



MARGINAL QUALITY DOCUMENT
BEST COPY AVAILABLE

DAVIS-BESSE NUCLEAR POWER STATION
TYPICAL WATERTIGHT DOOR
HINGE JAMB DETAIL
ALTERNATE EXISTING OPENING INSTALLATION
FIGURE 2.4-17

REVISION 0
JULY 1982

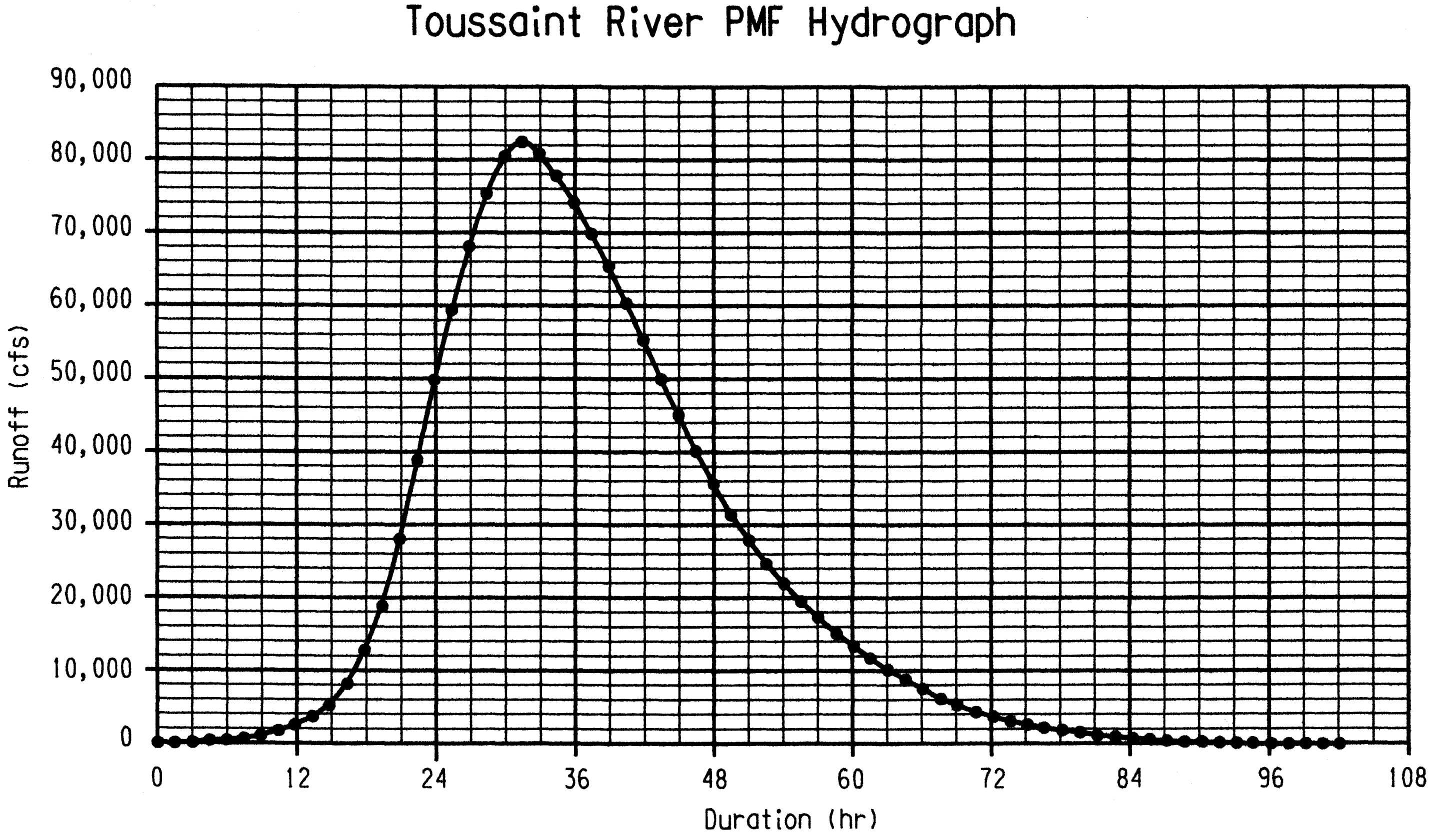


NOTE: THIS FIGURE IS DERIVED IN REFERENCE 62.

DAVIS-BESSE NUCLEAR POWER STATION
TOUSSAINT RIVER AT HIGHWAY 2 BRIDGE
3 HR. UNIT HYDROGRAPH
FIGURE 2.4-18

REVISION 30
OCTOBER 2014

DB 02-17-14 DFN: G/USAR/UFIG2418.DGN

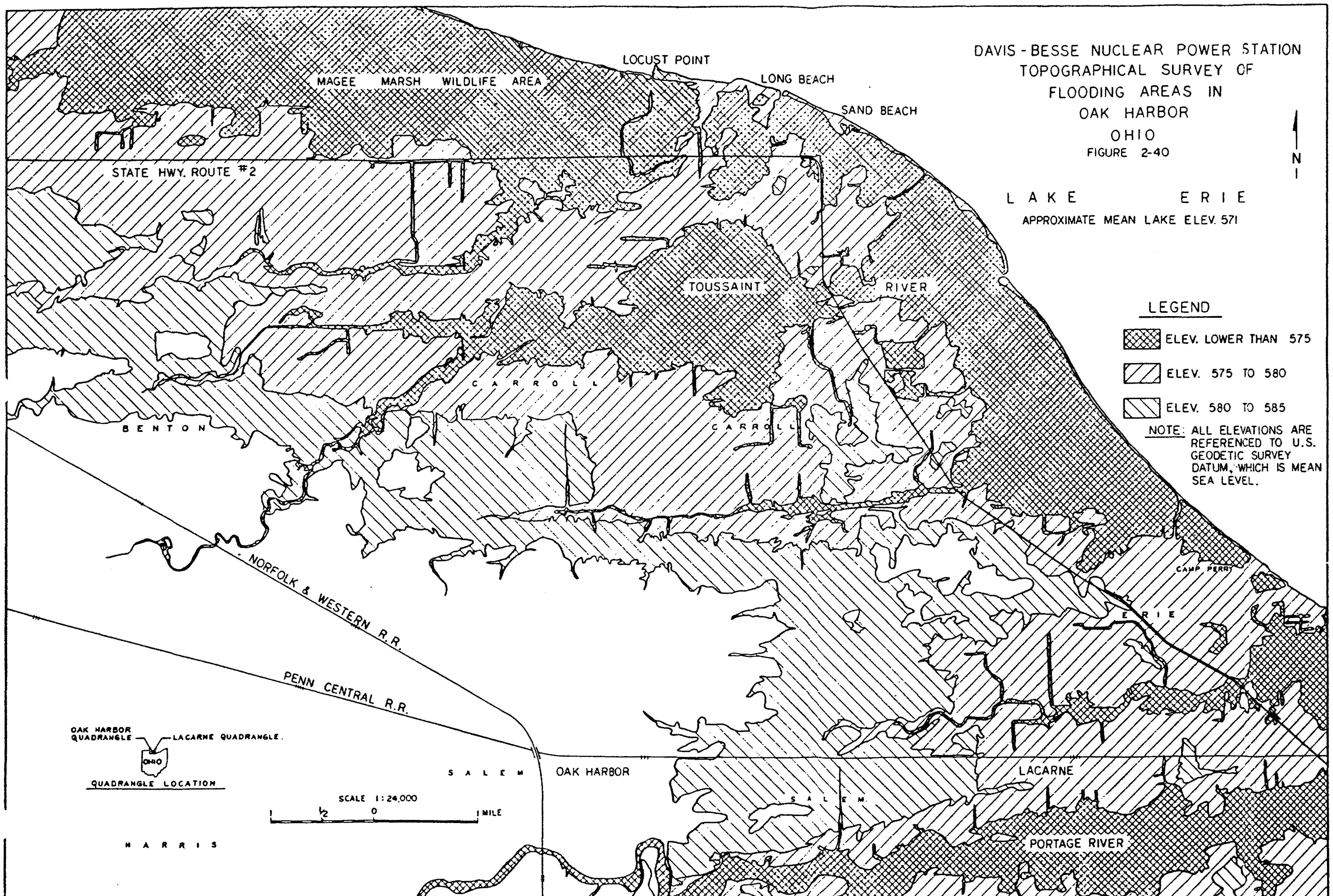


NOTE: THIS FIGURE IS DERIVED IN REFERENCE 62.

DAVIS-BESSE NUCLEAR POWER STATION
TOUSSAINT RIVER AT HIGHWAY 2 BRIDGE
TOTAL HYDROGRAPH (PMF)
FIGURE 2.4-19

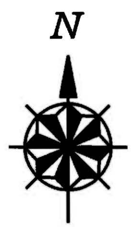
REVISION 30
OCTOBER 2014

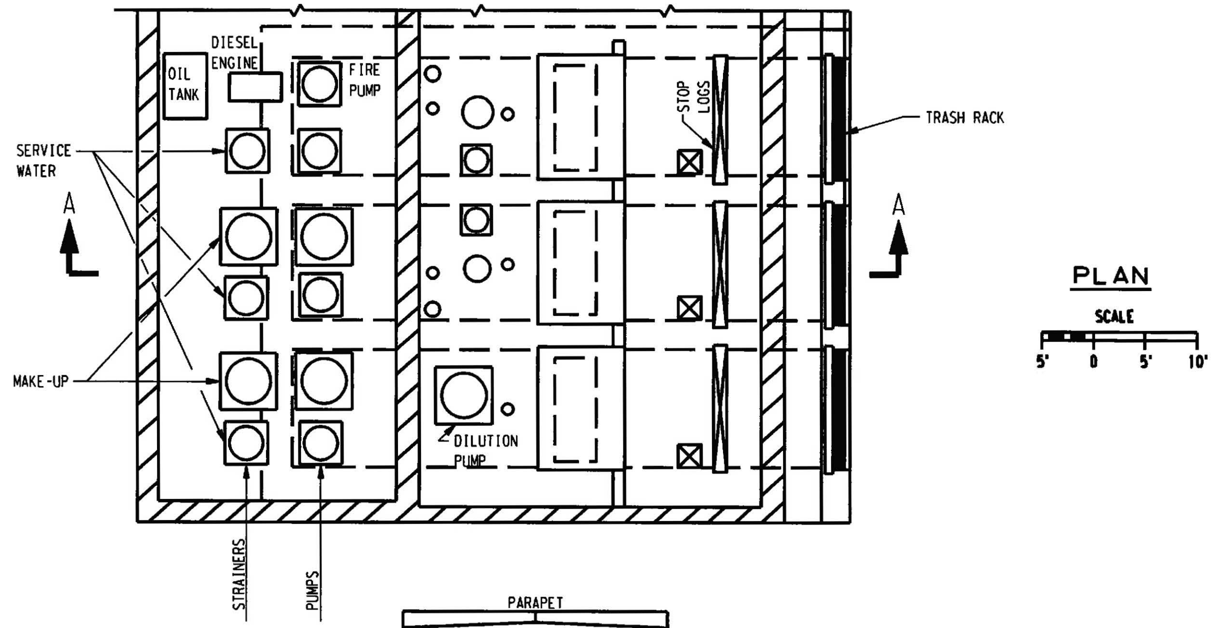
DB 02-17-14 DFN: G/USAR/UFIG2419.DGN



DAVIS-BESSE NUCLEAR POWER STATION
TOPOGRAPHICAL SURVEY OF
FLOODING AREAS IN
OAK HARBOR
OHIO
FIGURE 2.4-20

REVISION 0
JULY 1982

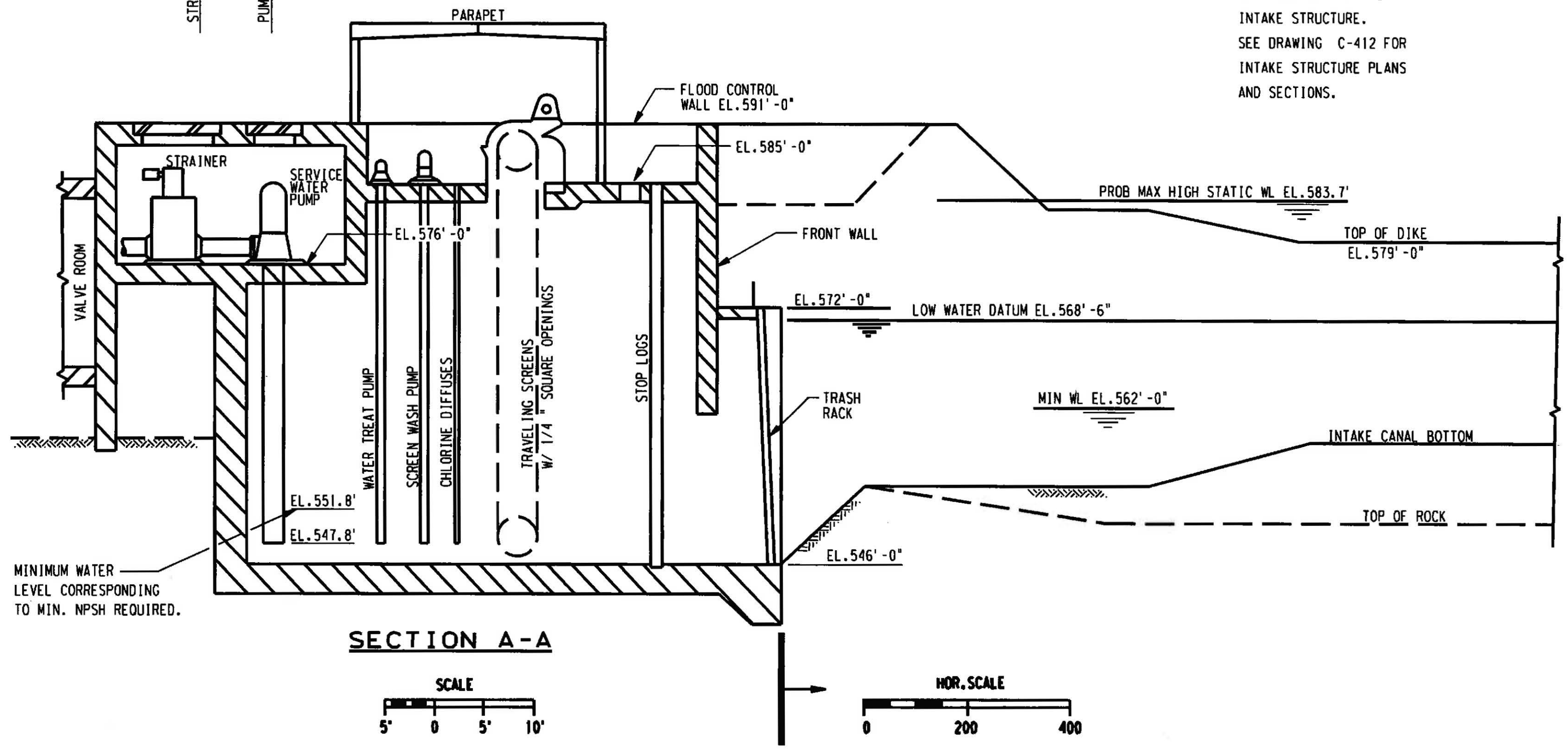




NOTES:

(1) ALL ELEVATIONS ARE REFERRED TO I.G.L.D.

(2) THIS FIGURE IS PROVIDED ONLY TO ILLUSTRATE WATER AND FLOOD CONTROL LEVELS WITH RESPECT TO THE INTAKE STRUCTURE. SEE DRAWING C-412 FOR INTAKE STRUCTURE PLANS AND SECTIONS.



DAVIS-BESSE NUCLEAR POWER STATION

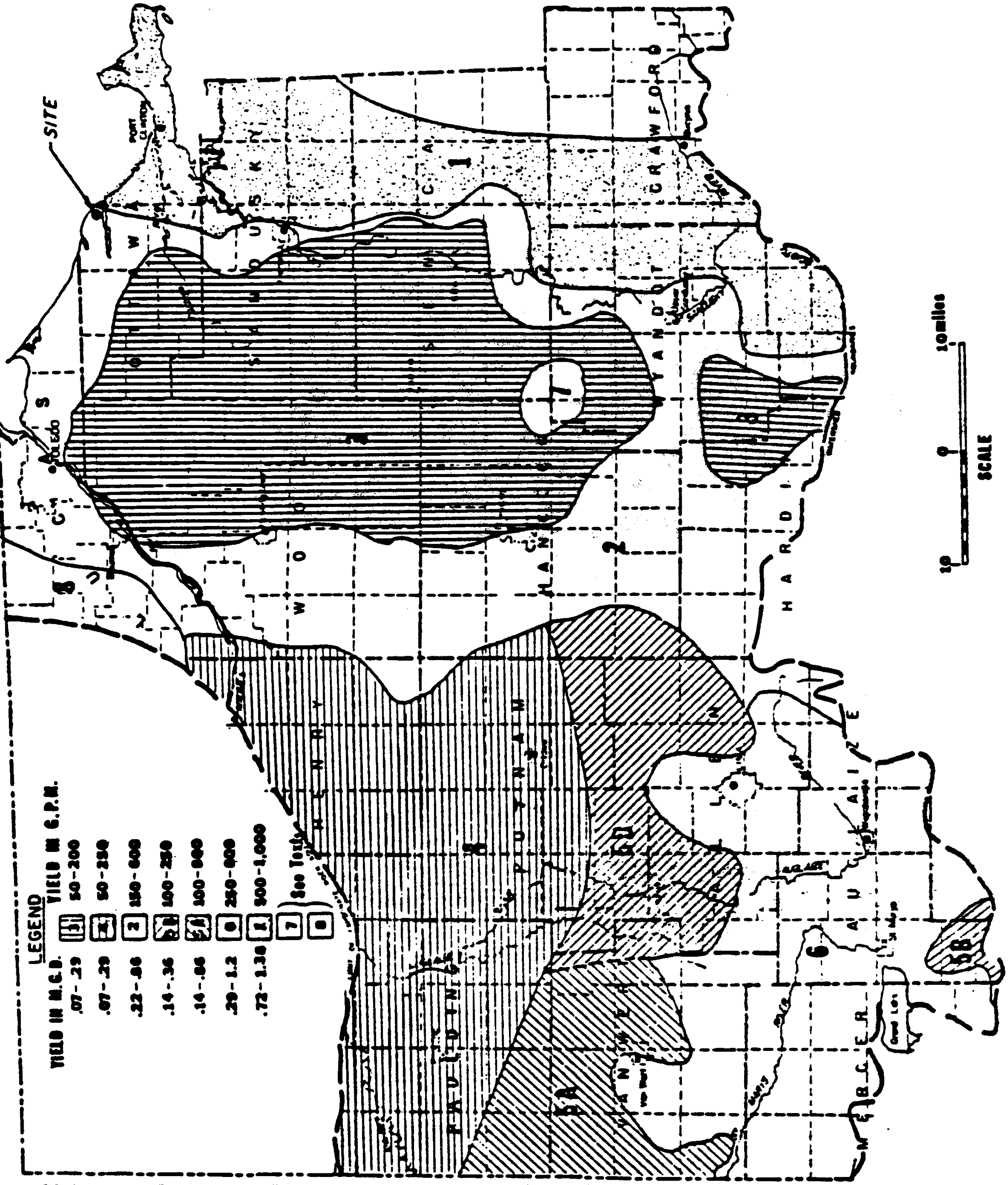
INTAKE STRUCTURE ARRANGEMENT

FIGURE 2.4-21

REVISION 31

OCTOBER 2016

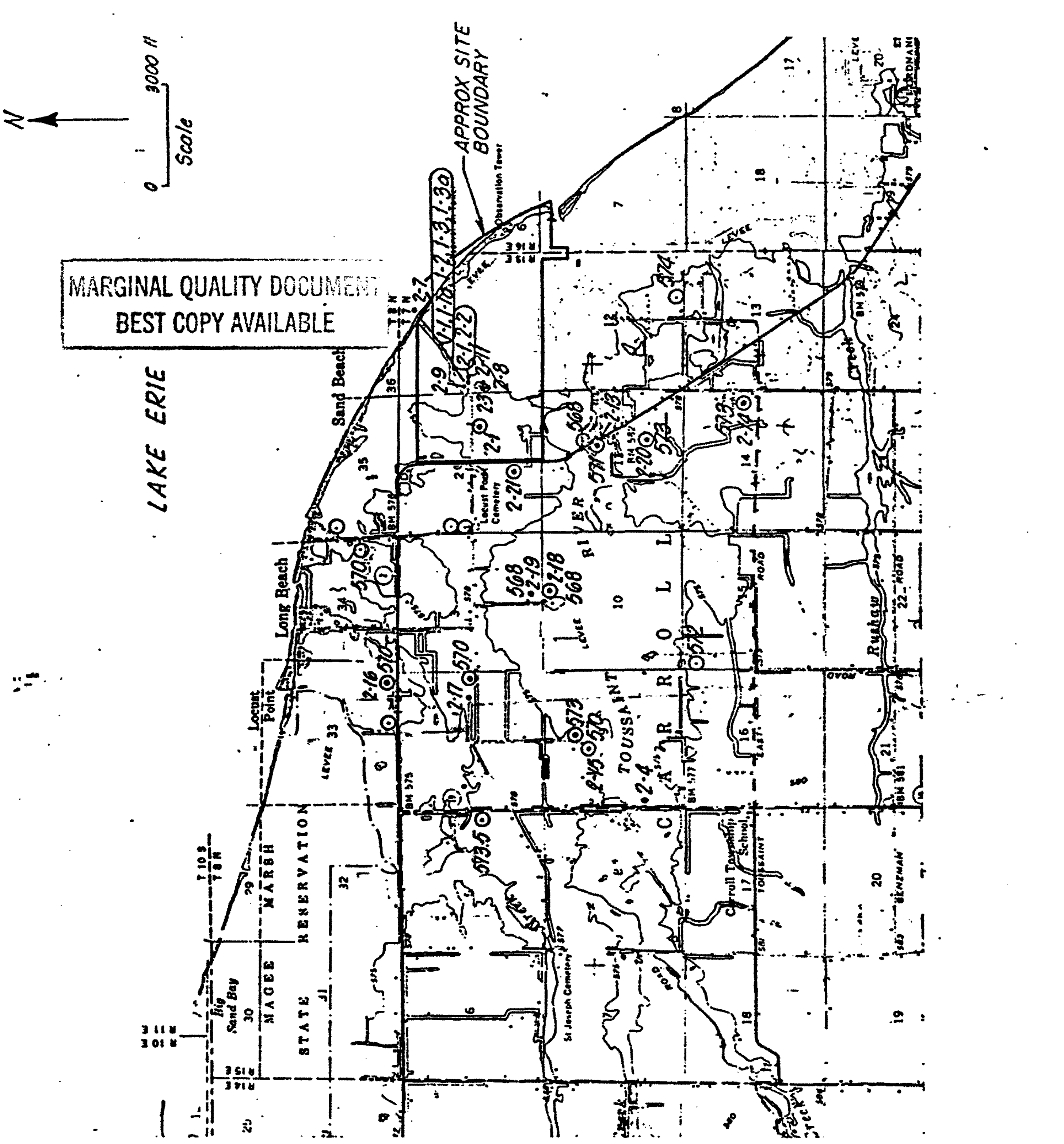
DB 03-18-16 DFN=G/USAR/FIG2421.DGN



MODIFIED AFTER OHIO WATER PLAN INVENTORY REPORT NO. 22, 1970, PREPARED BY OHIO DEPT. OF NATURAL RESOURCES.

DAVIS-BESSE NUCLEAR POWER STATION
WELL YIELDS DEVELOPED IN THE CARBONATE AQUIFER OF NORTHWESTERN OHIO
FIGURE 2.4-22

REVISION 0
JULY 1982



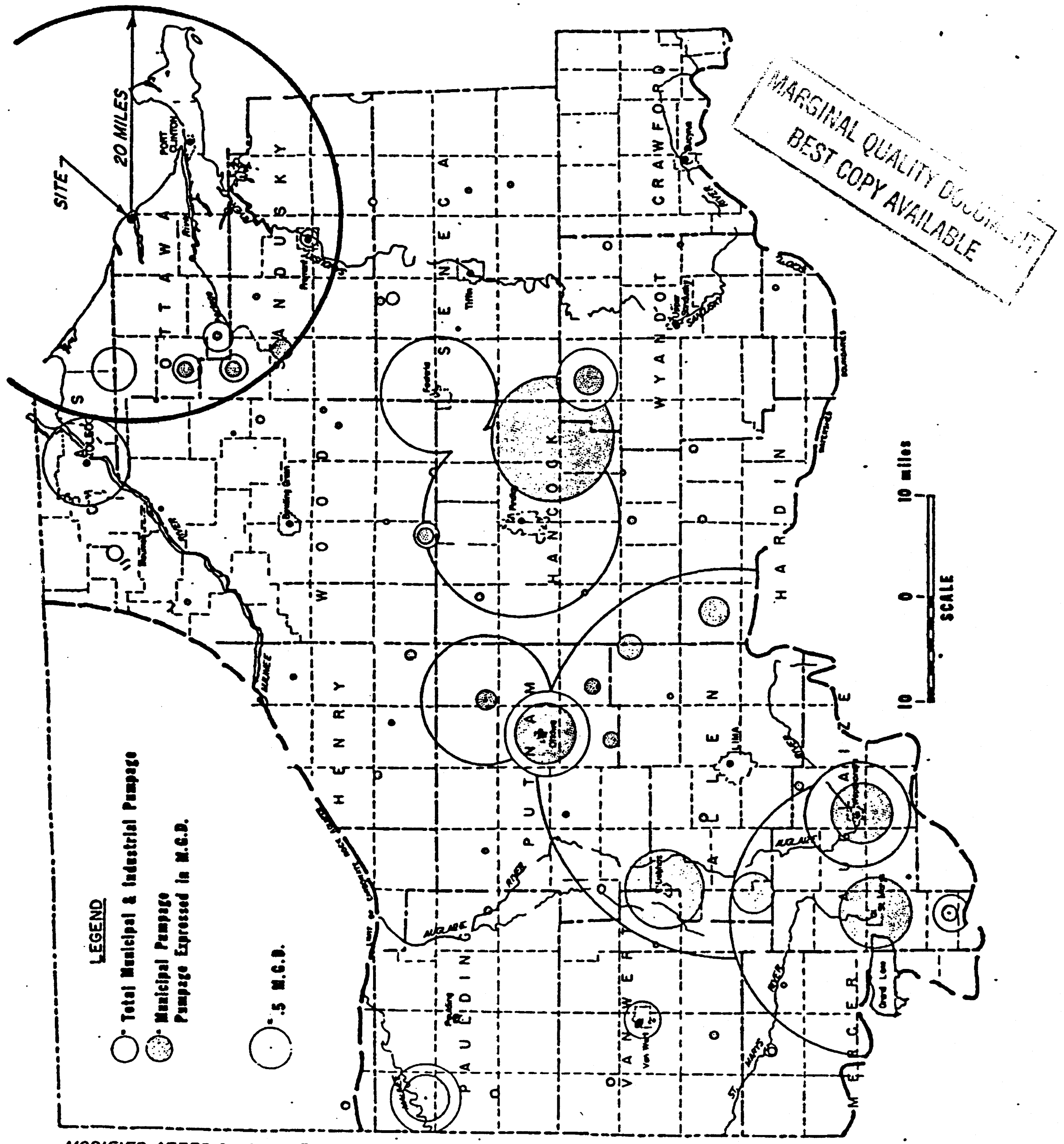
NOTES
1. Taken from USGS 15 min series topographic map of Oak Harbor Quadrangle (1952)
2. Ground surface contours referenced to USC&GS Datum. Approx water levels referenced to IGLD.

3. The following information pertains to the use of groundwater and was provided as part of the original safety analysis. Current conditions are not foreseen to differ significantly, however it is not practical to continuously update this section to reflect the current status. Therefore, this section is considered historical.

LEGEND
⊙ Location of well
570 Approx water level in wells (May 1969). Referenced to International Great Lakes Datum (IGLD).
←2-7 Location and number of water sample

DAVIS-BESSE NUCLEAR POWER STATION
LOCATIONS OF WELLS AND
WATER SAMPLES
FIGURE 2.4-23

REVISION 20
DECEMBER 1996

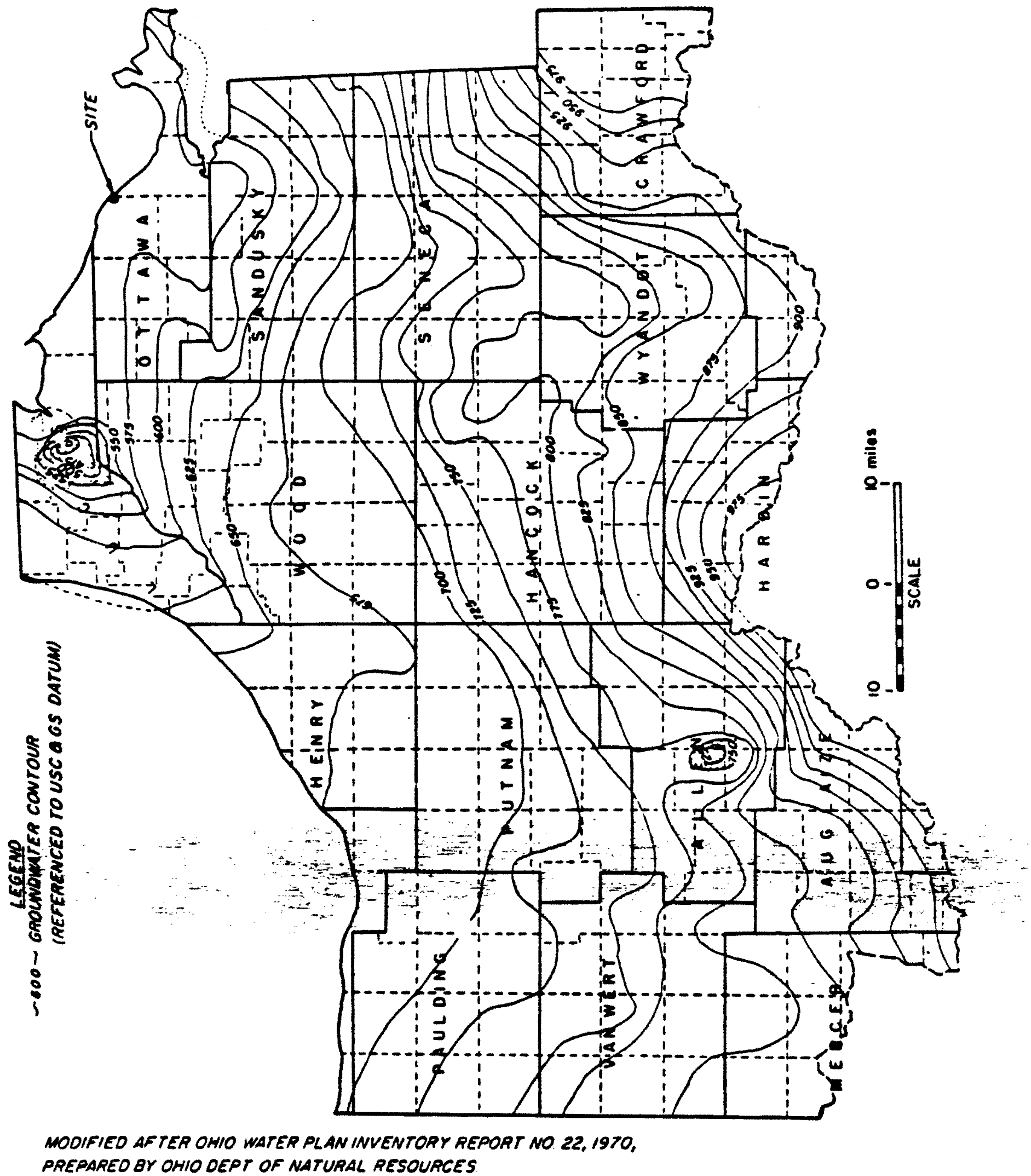


MODIFIED AFTER OHIO WATER PLAN INVENTORY REPORT NO. 22, 1970, PREPARED BY OHIO DEPT OF NATURAL RESOURCES.

Note:
The following information pertains to the use of groundwater and was provided as part of the original safety analysis. Current conditions are not foreseen to differ significantly, however it is not practical to continuously update this section to reflect the current status. Therefore, this section is considered historical.

DAVIS-BESSE NUCLEAR POWER STATION
PRESENT GROUNDWATER USAGE IN
CARBONATE AQUIFER OF
NORTHWESTERN OHIO
FIGURE 2.4-24

REVISION 20
DECEMBER 1996



MODIFIED AFTER OHIO WATER PLAN INVENTORY REPORT NO. 22, 1970,
PREPARED BY OHIO DEPT OF NATURAL RESOURCES

DAVIS-BESSE NUCLEAR POWER STATION
GROUNDWATER LEVEL CONTOURS IN
CARBONATE AQUIFER OF
NORTHWESTERN OHIO
FIGURE 2.4-25

REVISION 0
JULY 1982

2.5 GEOLOGY AND SEISMOLOGY

2.5.1 Basic Geologic and Seismic Data

2.5.1.1 General

Extensive geotechnical investigations were made of the site area and region. Geotechnical design criteria used in the design of station facilities were based on results of these investigations. The scope and results of the investigations and the geotechnical design criteria are presented and discussed in detail in Appendix 2C and are summarized in this section.

2.5.1.2 Regional and Local Physiography

The site region is located in the Lake Plains sub-province of the Central Lowland physiographic province. The Lake Plains sub-province is nearly flat and has poor surface drainage characteristics. The major streams in the region are the Maumee River, the Toussaint River (or Creek), and the Portage River; they generally flow toward the northeast and all drain into Lake Erie. These streams generally have a gradient of approximately 2 ft/mi which results in low velocities. The surficial soils are glacial deposits. Local sedimentary bedrock exposures have a very slight dip. The regional physiographic features are shown in Figure 2C.2-1 in Appendix 2C.

The site is situated on low, flat land bordering Lake Erie. The eastern portion of the site (approximately ½) is marshland at approximately 570 feet (I.G.L.D.); the western portion (approximately ½) is slightly above the marsh, at an elevation averaging 576 feet (I.G.L.D.). A narrow ridge of sand and man-placed riprap parallels the shoreline. Lake Erie low water datum is 568.6 feet (I.G.L.D.). The general site physiographic features are identified in Figure 2C.2-2 in Appendix 2C.

2.5.1.3 Regional Geologic and Tectonic Maps

Regional geologic maps are presented in Figure 2C.2-3 and 2C.2-4 in Appendix 2C. A regional geologic section is presented in Figure 2C.2-5 in Appendix 2C.

Regional tectonic maps are presented in Figure 2C.2-3 and 2C.3-1 in Appendix 2C.

2.5.1.4 Site Geologic Mapping

The site surface geology map is presented in Figure 2C.4-5 in Appendix 2C. Geologic maps of bedrock at foundation grade were prepared and are shown in Figures 2C.5-21 and 2C.5-22 in Appendix 2C. Bedrock surface contours at foundation grade are shown in Figure 2C.5-35 in Appendix 2C. Generalized geologic profiles showing the relationship of the major foundations to the subsurface conditions are presented in Figures 2C.4-6 through 2C.4-8 in Appendix 2C.

2.5.1.5 History of Groundwater Fluctuations

Prior to construction, the groundwater level was approximately 571 to 572 feet (I.G.L.D.). (This is 1-2 feet above the average Lake Erie water level.) Seasonal fluctuations of the piezometric head in the bedrock aquifer are small and on the order of 2 feet to 5 feet. Groundwater levels range from 570 to 575 feet (I.G.L.D.). In the station area, the elevation of the bedrock surface is on the order of 10 feet below the average Lake Erie water level. Because of the low permeability of the overlying glacial deposits, the groundwater in the more pervious bedrock aquifer is confined under a piezometric head of approximately 10 feet.

During construction, the water level was lowered by a construction dewatering system to 525 feet (I.G.L.D.) in the Containment Building area. The maximum radius of influence of this dewatering system was approximately 5500 feet. A complete description of the groundwater monitoring program performed during construction is presented in Section 2C.5 of Appendix 2C.

In late summer 1972, the construction dewatering operations were discontinued. No pumping of groundwater is contemplated during operation of the power station. On completion of the excavation, water returned to normal levels within a year after dewatering operation was finished.

2.5.1.6 Subsurface Investigations

The preconstruction and construction phase subsurface investigations are discussed in Appendix 2C. These investigations consisted of 125 borings, 184 soil probes, 88 6-in.-diameter rock probes, 156 3-in.-diameter rock probes, 142 seismic refraction measurement, 91 cross-hole seismic compression wave measurements, 459 gravity survey measurements, 275 resistivity survey measurements, 148 in-situ bedrock permeability tests, geologic mapping of natural and excavated bedrock surfaces, two test pits, and three sections of a test excavation. The locations of these investigations measurements made in the station area are presented in Figures 2C.4-2, 2C.4-3, 2C.5-3, 2C.5-21, 2C.5-22, 2C.5-23, 2C.5-25, 2C.5-29, 2C.5-34, and 2C.5-37 in Appendix 2C.

Based on the analysis of the results obtained from these investigations, it is concluded that bedrock in the station area is free of significant solution activity and is a competent foundation material. Results of all investigations in the station area are presented and analyzed in Appendix 2C.

2.5.1.7 In-Situ Compression and Shear Wave Velocity Measurements in the Station Area

Twenty-six seismic refraction shot points were used to determine the in-situ shear wave (S-wave) velocity of the bedrock in conjunction with 26 shots made for compression wave (P-wave) velocity measurements.

The in-situ S-wave velocity of the soil deposits (till and glaciolacustrine deposits) could not be measured by surface refraction techniques because S-wave arrivals were masked by P-waves.

A discussion of the procedures used and results obtained from these in-situ measurements is presented in Appendix 2C. Typical ranges of P-wave and S-wave velocities measured in the station area are summarized as follows:

	P-Wave Velocity ft/sec	S-Wave Velocity ft/sec
Till Deposit	5,100 to 6,100	
Bedrock	11,400 to 14,600	5,700 to 7,500

2.5.1.8 Summary of Static and Dynamic Soil and Bedrock Properties

2.5.1.8.1 Soil Deposits

Major soil deposits at the site consist of a glaciolacustrine and a till deposit. The glaciolacustrine deposit consists of stiff, fissured, gray and brown silty clay. Representative values for selected static properties for the glaciolacustrine deposit are listed as follows:

Property	Representative Value*
Range of thickness in station area, ft	6 to 10
Water content, %	24
Liquid Limit	51
Plastic Limit	23
Unit weight, lb/cubic foot	125
Unconfined compression strength, tons/sq ft	3.5
Standard penetration resistance, blow/ft	12
Permeability, cm/sec	less than 10^{-6}
Compression Index, C_c	0.15
Recompression index, C_r	0.004
Coefficient of consolidation, C_v sq.cm/sec	0.5×10^{-2}
Range of maximum past effective consolidation pressure, tons/sq foot	4 to 12

* No dynamic parameters were determined for glaciolacustrine deposit because no major structures were founded on the deposit.

The till deposit consists of a hard, fissured, gray to brown silty, sandy clay with generally less than 10 percent gravel. Representative values for selected static and dynamic properties for till deposit are listed as follows:

Property	Representative Value
Range of thickness in station area, ft	6 to 10
Water content, %	15
Liquid Limit	33
Plastic Limit	17
Unit weight, lb/cu. ft	132
Unconfined compression strength, tons/sq. ft	8
Standard penetration resistance, blows/ft	40
Permeability, cm/sec	less than 10^{-6}
Compression, index, C_c	0.08
Recompression index, C_r	0.02
Coefficient of consolidation, C_v sq cm/sec	1×10^{-2}
Range of maximum past effective consolidation pressure, tons/sq. ft	10 to 50
Compression modules, E, 10^3 kips/sq. ft.	28 to 34*
Shear modulus, G, 10^3 kips/sq. ft.	10 to 12*
Poisson's ratio, μ	0.4
Damping radio, λ	0.04 to 0.05**

*Values are strain dependent. Smallest value used with Maximum Possible (larger) Earthquake; largest value used with Maximum Probable (smaller) Earthquake.

**Values are strain dependent. Largest value used with Maximum Possible (larger) Earthquake; smallest value used with Maximum Probable (smaller) Earthquake.

2.5.1.8.2 Bedrock Formation

The bedrock formation is the Tymochtee formation which consists of argillaceous dolomite with inter-bedded gypsum, anhydrite, and shale strata. The argillaceous dolomite can be divided into two major units; a massive dolomite and a bedded dolomite. A description of each dolomitic rock unit and representative static and dynamic properties follows.

The massive dolomite occurs in an 8-ft- to 10-ft-thick stratum, the top of which is located approximately 10 ft below the bedrock surface. The massive dolomite is medium hard to hard, buff to gray, and argillaceous.

The bedded dolomite occurs above and below the massive dolomite unit. It is medium hard, gray to buff, and argillaceous with frequent laminae of gypsum, anhydrite, and shale.

Representative static and dynamic properties of bedrock are listed as follows:

Property	Representative Value
Unit weight, lb/cu. ft.	150
Unconfined compression strength, tons/sq. ft.	800
Compression modulus, E, 10^3 kips/sq. ft.	390 to 470*
Shear modulus, G, 10^3 kips/sq. ft.	150 to 180*
Poisson's ratio	0.3
Damping radio	0.01 to 0.02**

* Values are strain dependent. Smallest value used with Maximum Possible Earthquake; largest value used with Maximum Probable Earthquake.

**Values are strain dependent. Largest value used with Maximum Possible Earthquake; smallest value used with Maximum Probable Earthquake.

2.5.1.9 Seismic Class I Earthwork Construction

2.5.1.9.1 General

Seismic Class I earthwork construction included construction of dikes along the intake forebay canal from approximately station 0+00 to approximately station 7+00 and the construction of backfills to support some Class I structures in the station area. Geotechnical criteria established for construction of these Class I fills are presented in Section 2C.6 of Appendix 2C.

2.5.1.9.2 Criteria for Seismic Class I Intake Forebay Dike Fill

Seismic Class I intake forebay dike fill (hereafter referred to as Class I intake fill) consists of compacted glaciolacustrine and till deposits obtained from on-site borrow areas.

Class I intake fill was to be placed in lifts with a maximum thickness of 12 in. and compacted with a minimum of six coverages of an approved sheepfoot roller. Class I intake fill material was to be placed and compacted within a water content range of in-situ water content to optimum water content minus 5 percentage points. Figure 2C.6-1 in Appendix 2C shows the approximate extent of Class I intake fill. Figure 2C.6-2 in Appendix 2C shows a typical profile of the intake forebay canal dike.

2.5.1.9.3 Criteria for Seismic Class I Structural Backfill in the Station Area

Seismic Class I structural backfill in the station area (Class I backfill) consists of crushed rock obtained from the onsite quarry stockpile (granular fill material). The approximate gradation characteristics of the granular fill material are shown below:

Sieve Size	Total Percent Passing
2 ½ inch	100
1 inch	60-100
No. 100	0-30
No. 200	0-5

Class I backfill was to be placed in loose lift thicknesses ranging from 15 in. in large work areas[1] to 6 in. in small work areas[2]. Structural backfill was to be compacted to 98 percent of the maximum dry density determined in accordance with ASTM Specification No. D698 Method D, latest revision or to 80 percent relative density determined in accordance with ASTM Specification No. D-2049.

2.5.1.10 Geotechnical Design Criteria for Seismic Class I Station Facilities

2.5.1.10.1 General

Seismic Class I station facilities include some station area structures and a portion of the intake forebay canal dikes. The geotechnical design criteria established for the design of foundations for Class I structures and for the construction of intake forebay dikes, are presented in Section 2C.6 of Appendix 2C. A summary of the criteria and a discussion of the factors of safety these criteria represent in terms of the stability of Class I station facilities is presented below.

2.5.1.10.2 Foundations for Seismic Class I Structures

Foundations for Class I station structures consist of mat or strip footings bearing on bedrock, till deposit, or compacted granular fill and pier footings socketed into bedrock.

The maximum design bearing capacity, ultimate bearing capacity, and ultimate bedrock socket-concrete bond strength for geologic materials supporting foundations for Class I structures are as follows:

Bearing Material	Maximum Design Bearing Capacity k/sq ft	Maximum Design Bedrock Socket-Concrete Bond Strength k/sq ft
1. Bedrock free of significant solution activity	100	36
2. Granular structural fill constructed on till deposit	10	NA
3. In-situ till deposit	10	NA

Table 2.5-1 shows the maximum contact stress expected beneath major strip footing foundations supporting Class I structures, the ultimate bearing capacity of geologic materials beneath these footings, and the expected maximum settlement of the footing at the maximum contact stress.

The factor of safety of mat and strip footing foundations for Class I structures against a bearing capacity failure (expressed as a ratio between the ultimate bearing capacity of material beneath the footing and the maximum contact stress beneath the footing) is greater than 5. Total settlement of Class I structures founded on bedrock will be less than 1/10 in., and total settlement of Class I structures founded on till deposit and granular fill will be less than 1/4 in. Settlement of structures will be elastic within the range of footing contact stresses anticipated.

[1]Large work areas are defined as work areas where compaction is accomplished with a minimum of 4 coverages of a Raygo 400, or approved equivalent compactor.

[2]Small work areas are defined as work areas where compaction is accomplished with a minimum of 4 coverages of a Super Ground Pounder Model 7000, or approved equivalent compactor.

Consequently, settlements will occur upon application of footing stresses and no long-term settlement of structures is expected.

Socketed Pier Footing Foundations:

A portion of the Auxiliary Building is supported on socketed pier footings foundations. The maximum load expected on these footings is 1700 kips. The ultimate load that can be supported by the bedrock socket is 4900 kips. Settlement of individual piers, at the 1700 kip load, is expected to be less than 1/4 inch.

Seismic Class I Intake Forebay Canal Dikes:

The Class I intake forebay canal dikes extend along the intake canal from approximately station 0+00 to approximately station 7+00. Placement and compaction of Class I dike fill were in accordance with the requirements of project specification and within the dimensional limits shown in design drawings.

The Reference 59 evaluation was made to determine the factor of safety of the Class I intake canal dikes during the Maximum Possible Earthquake. The method of analysis and the assumed conditions are discussed in Subsection 2.5.5.

Results of the stability analysis indicated that the Class I intake canal dikes have a minimum factor of safety of 1.30 against failure during the Maximum Possible Earthquake.

2.5.2 Vibratory Ground Motion

2.5.2.1 General

Extensive geologic and seismologic investigations were made of the site area and region. Dynamic parameters for use in the design of station facilities were recommended based on results of these investigations. The scope and results of the investigations and the recommended dynamic parameters used in the design are presented and discussed in detail in Appendix 2C and are summarized in this section.

2.5.2.2 Description of Lithographic, Stratigraphic, and Structural Geologic Conditions

The regional and local lithographic, Stratigraphic, and structural geologic conditions are described in detail in Section 2C.2 of Appendix 2C.

2.5.2.3 Identification of Tectonic Structures

The major tectonic structures in the region are the Findlay Arch, the Michigan Basin, the Appalachian Geosyncline, the Ohio-Indiana Platform, and three faults: the Bowling Green fault, the Electric Fault, and the Osborn Fault. The locations of these structures are shown in Figure 2C.2-3 of Appendix 2C.

The site is located on the east flank of the Findlay Arch. No other tectonic structures are known to underlie the site. The nearest fault is the Bowling Green Fault, which is located 35 miles west of the site. A complete discussion of these tectonic structures is presented in Section 2C.2 of Appendix 2C.

2.5.2.4 Physical Evidence Concerning Behavior of Soil Deposits and Bedrock at Site During Prior Earthquakes

Results of the geologic, seismologic, and subsurface investigations indicate no evidence of fault traces, offset geomorphic features, shear zones, faults, sand boils, soil flows, or any other direct or indirect physical effects of prior earthquakes.

2.5.2.5 Summary of Static and Dynamic Engineering Properties of Soil Deposits and Bedrock Required for Seismic Design

Subsection 2.5.1.7 and 2.5.1.8 summarized all static and dynamic engineering properties of soil deposits and bedrock at the site required for a seismic design of the nuclear power station. Backup data for these properties are presented in Section 2C.4 of Appendix 2C.

2.5.2.6 Historic Earthquakes

A detailed study of historic earthquakes which have or may have affected the site is presented in Section 2C.3.2 of Appendix 2C. Table’s 2C.3-1, 2C.3-2, and 2C.3-3 of Appendix 2C present data for all pertinent historic earthquakes. Based on the study of historic earthquakes, it is believed that no historic earthquake has caused acceleration greater than 0.02g in soil deposits and bedrock underlying the site.

Table’s 2C.3-1, 2C.3-2, and 2C.3-3 in Appendix 2C present estimates of Richter Magnitudes for historic earthquakes in the site region. These estimates are based on two correlations.

A relationship between Richter Magnitude and epicentral intensity has been proposed by Messrs. Gutenberg and Richter (ref. 2).

$$M = 1.3 + 0.6\, I_0$$

Where:

M = Richter Magnitude

I_0 = Epicentral Intensity

The relationship is strongly based on California earthquakes and would not necessarily be accurate for central United States earthquakes.

The second calculation was proposed by Housner (ref. 1) and relates felt areas of earthquake with Richter Magnitude.

Housner’s relationship for “central region” of the United States is:

$$M = 2.3 \log_{10}(A+14{,}000)-6.6$$

Where:

M = Richter Magnitude

A = felt area in square miles

This second relationship was developed by Housner based on data for the central United States and should be applicable to the Davis-Besse site region. However, all historic earthquakes listed in Tables 2C.3-1, 2C.3-2, and 2C.3-3 do not have reported felt areas, and thus Richter Magnitudes could not be calculated by the Housner relationship for all earthquakes in these tables.

2.5.2.7 Correlation of Earthquake Epicenters within 200 Miles of Site with Tectonic Structures and Provinces

Correlation of historic earthquake epicenters with tectonic structures of provinces is presented in Sections 2C.3.2 and 2C.3.3 of Appendix 2C. Figures 2C.3-2 and 2C.3-3 present the locations of both earthquake epicenters and tectonic structures and provinces pertinent to the site.

2.5.2.8 Active Faults

Based on the results of the geologic, seismologic, and subsurface investigations, it is concluded that there is no fault within 200 miles of the site which is considered active or capable of causing surface displacements. Sections 2C.2, 2C.3, and 2C.4 of Appendix 2C present data to support this conclusion.

2.5.2.9 Correlation of Greatest Historic Earthquakes with Tectonic Structures and Provinces

The largest historic earthquake associated with tectonic structures and provinces are presented in Sections 2C.3.2 and 2C.3.3 of Appendix 2C. Section 2C.3.4 of Appendix 2C presents the methods used to determine the Maximum Possible (larger) Earthquake and Maximum Probable (smaller) Earthquake. The determination of the Maximum Possible Earthquake is made assuming the largest historic earthquake associated with the tectonic structure would occur at the closest point of this structure to the site.

2.5.2.10 Maximum Possible (Larger) Earthquake

The selection of the Maximum Possible Earthquake is presented in Section 2C.3.4 of Appendix 2C. Table 2C.3-4 of Appendix 2C presents the recommended parameters of the Maximum Possible Earthquake; Figure 2C.3-5 presents the recommended response spectra; and Table 2C.3-6 lists the recommended amplification factors for various frequency ranges.

2.5.2.11 Maximum Probable (Smaller) Earthquake

The selection of the Maximum Probable Earthquake is presented in Section 2C.3.4 of Appendix 2C. Table 2C.3-4 of Appendix 2C presents the recommended parameters of the Maximum Probable Earthquake; Figure 2C.3-6 presents the recommended response spectra; and Table 2C.3-6 lists the recommended amplification factor for various frequency ranges.

2.5.3 <u>Surface Faulting</u>

Based on the results of the geologic, seismologic, and subsurface investigations 2C.2, 2C.3, and 2C.4 of Appendix 2C, it was concluded that there are no active faults nor surface faulting phenomena at or near the site, nor is there potential for such faulting. There are no faults known or suspected to exist within 5 miles of the site. The nearest fault is the inactive Bowling Green fault located 35 miles west of the site. Because of the absence of any faulting in the site

area or region, the need to consider surface faulting in the design of station was considered unnecessary.

2.5.4 Stability of Subsurface Materials

2.5.4.1 Surface or Subsurface Subsidence

2.5.4.1.1 Solution Activity in Bedrock

A minor portion of the bedrock formation at the site, in particular gypsum strata which account for approximately 20 percent of the bedrock formation above 500 feet (I.G.L.D.), is known to be susceptible to solution activity under certain groundwater conditions. For this reason, a bedrock verification program was implemented during formation construction to determine the degree of solution activity, if any, in bedrock in the station area.

The results of the Bedrock Verification Program (BVP) are presented in Section 2C.5 of Appendix 2C. No evidence of significant solution activity was indicated by results of the BVP, and it is concluded that no significant solution activity exists in the investigated portion of the station area. Consequently, surface or subsurface subsidence of bedrock in the station area caused by the collapse of solution cavities or fissures during vibratory motion associated with the Maximum Possible Earthquake is not considered as a reasonable possibility.

2.5.4.1.2 Mineral Extraction

There has been no subsurface mining activities in the site area nor is there believed to be economic potential for such mining in the future. Consequently, there is no potential for surface or subsurface subsidence at the station caused by the collapse of subsurface mining facilities during vibratory motion associated with the Maximum Possible Earthquake.

2.5.4.1.3 Subsurface Fluid Addition or Withdrawal

There will be no water wells or other withdrawal of groundwater due to pumping during the operation of the station. Very few oil and gas wells are located in the site locality. Operation of these wells has no significant effect on the stability of subsurface materials at the site.

There are no other known significant subsurface fluid addition or withdrawal operations in the vicinity of the site (i.e., hot water salt mining, municipal water wells, etc.). Subsection 2.4.13.2 presents a discussion of groundwater usage in the site region.

Based on the results of the study of subsurface fluid addition or withdrawal, it is concluded that there are no significant cavities or fissures beneath the site caused by such additions or withdrawals which would result in surface or subsurface subsidence during the vibratory motions associated with the Maximum Possible Earthquake.

2.5.4.1.4 Regional Warping

The site is located in the Great Lake Basin. This basin appears to be undergoing minor isostatic adjustment resulting from the retreat of the last glacial loadings. The results of the geologic and seismic studies indicated seismic activity in the site locality has not been associated with these adjustments. At the site, isostatic adjustments amount to a slow subsidence of the area. The rate of subsidence in the site region is estimated to be 0.5 to 1.0 ft/100 years (see Section 2C.2 of Appendix 2C for a complete discussion of the nature of these adjustments). Because of the

essentially rigid body characteristics of these movements, their relatively elastic nature, and the lack of any structural weakness in the bedrock beneath the site, it is concluded that there is no potential for these isostatic adjustments to result in surface or subsurface subsidence during vibratory motion associated with the Maximum Possible Earthquake.

2.5.4.2 Shear Zones, Joints, Fractures, Folds, Zones of Alteration, Irregular Weathering, or Structural Weakness

Based on the results of the geologic, seismologic, and subsurface investigations, it is concluded that the bedrock beneath the foundations of the nuclear power station is free of significant zones of structural weakness. No shear zones, zones of major folding, zones of alteration, or zones of irregularly weathered bedrock were detected in bedrock beneath foundations for station area structures; Sections 2C.2, 2C.4, and 2C.5 of Appendix 2C present further discussion of this topic.

The joint and natural fractures observed in the station area test borings and foundation excavations were typically healed with calcite or satin spar gypsum and were no wider than 0.1 ft; see Section 2C.5 of Appendix 2C.

Because there are no significant zones of structural weakness in the bedrock underlying the station, there is no potential for such zones of weakness to cause instability during the vibratory motion associated with the Maximum Possible Earthquake.

2.5.4.3 Unrelieved Residual Stresses in Bedrock

Based on results of the geologic, seismologic, and subsurface investigation, it is concluded that there are no significant unrelieved residual vertical or horizontal stresses in the bedrock in the station area. Because there are no significant unrelieved residual stresses in the bedrock at the site, there is no potential for such stresses to cause instability during the vibratory motion associated with the Maximum Possible Earthquake.

2.5.4.4 Anhydrite-Gypsum

The bedrock formation below elevation 500 contains approximately 40 percent of the anhydrite. Under certain temperature, pressure, and groundwater conditions, anhydrite can be transformed into gypsum with an increase in volume as a result of addition of water of hydration to the structure. However, under the temperature, pressure, and groundwater conditions at the site, the anhydrite is considered stable and will remain stable.

2.5.4.5 Seismic Class I Granular Backfill

The Class I granular backfills in the station area are founded on the dense glacial till deposit and/or bedrock. Based on the gradation of the backfill material and placement criteria, Class I granular backfill will not undergo differential consolidation or liquefaction under the effects of the vibratory motion of the Maximum Possible Earthquake. Consequently, there is no potential for these earthquake induced phenomena to cause instability of the Class I granular backfill beneath the foundations of the station.

Because the granular fill material is obtained from an onsite quarry, it contains approximately 20 percent (by weight) of gypsum, the potential for solution of gypsum in the material was determined in an extensive series of laboratory tests.

Based on results of these tests, it was concluded that under existing and anticipated future groundwater gradients, temperature, and chemical conditions, the granular backfill material is not susceptible to solution that could result in instability of foundations of Class I structures during vibratory ground motions associated with the Maximum Possible Earthquake.

2.5.5 Slope Stability

2.5.5.1 General

The stability of soil slopes for the Class I intake canal dikes was analyzed to determine the factor of safety against failure during the occurrence of a Maximum Possible Earthquake.

2.5.5.2 Method of Analysis and Results

The extended duration condition is associated with the dike slope stability after a period of time during which the groundwater levels (pore-water pressures) within the dike reach an equilibrium condition with the water level inside the canal and the surrounding land around the dike. The length of this period may vary from a few months for relatively permeable soils to several years for low-permeability soils. This is the condition addressed in the Reference 59 evaluation since the dikes were constructed approximately three decades ago.

The factor of safety is defined as the ratio of the available undrained shear strength on an assumed failure to the undrained shear strength along the same failure surface required to provide a factor of safety of 1.0 for assumed loading conditions.

For purposes of the analysis, the water level in the intake area was assumed to be at 576 feet (I.G.L.D.); a horizontal force equal to 0.15 times the weight of the sliding mass was assumed to act in the direction of sliding as documented in Reference 59.

Factors of safety were calculated for several assumed failure surfaces to determine the critical failure surface which would result in the lowest factor of safety. Circular arc and sliding wedge modes of failure were analyzed to determine the required undrained shear strength along the failure surface for the assumed loading conditions. The sliding wedge mode of failure resulted in the minimum factor of safety. As documented in the Reference 59 evaluation, the sliding wedge failure remains the appropriate failure mode as dictated by the soil properties. Figure 2C.6-2 in Section 2C.6 of Appendix 2C shows the dike cross section, material distribution and water level for the seismic loading conditions. Results of the Reference 59 analysis indicate that the slope stability of the dike for the sliding wedge mode of failure for the current (as-built) condition is acceptable.

The undrained shear strength characteristics of the compacted glaciolacustrine and till deposits were determined in two series in strength tests. Prior to construction, a series of Unconsolidated-Undrained (UU) triaxial tests were made on samples of compacted glaciolacustrine and till deposit (compaction was in accordance with ASTM Specification No. D698-68T Method A).

The UU tests indicated that the undrained shear strength of compacted glaciolacustrine deposit ranged from approximately 2.6 kips/sq. ft. at a water content of 20 percent to 1.0 kips/sq. ft. at a water content of 25 percent; and that the undrained shear strength of compacted till deposit ranged from 5.0 kips/sq. ft. at a water content of 13 percent to 0.6 kips/sq. ft. at a water content of 19 percent. It is concluded, on the basis of test procedures used to make the UU tests and on the basis of the plasticity indices of the material tested, that the data obtained represent the

lower bound of undrained shear strength of compacted glaciolacustrine and till deposits. Reference 59 indicates that the soil parameters used to evaluate soil stability are consistent with this range of parameters.

During construction, a series of thirteen in-situ vane shear strength tests were made in the Class I fill at the four locations shown in Figure 2C.6-1. A discussion of these tests is presented in Section 2C.5 of Appendix 2C. These tests indicated that the vane shear strength of compacted glaciolacustrine deposits ranged from approximately 4.0 kips/sq. ft. at a water content of 20 percent to 2.0 kips/sq. ft. at a water content of 29 percent; and that the vane shear strength of compacted till deposit ranged from 7.0 kips/sq. ft. at a water content of 15 percent to 3.5 kips/sq. ft. at a water content of 19 percent. It is concluded, on the basis of test procedures used, that the data obtained from the vane tests represent the upper bound of undrained shear strength of compacted glaciolacustrine and till deposit. Reference 59 indicates that the soil parameters used to evaluate soil stability are consistent with this range of parameters.

Water content tests were made on samples of the in-place fill obtained during inspection of placement and compaction of Class I fill.

Ninety-seven water content tests made on till deposit samples indicated the average water content of till deposit placed and compacted as Class I fill was approximately 16 percent. One hundred forty-four water content tests made on glaciolacustrine deposit samples indicated the average water content of glaciolacustrine deposit placed and compacted as Class I fill was approximately 25 percent.

Reference 59 evaluates the sliding wedge mode of failure and indicates a minimum factor of safety of 1.30 for the Class I dikes during application of Maximum Possible Earthquake forces.

TABLE 2.5-1

Summary of Maximum Contact Stresses and Ultimate Bearing Capacity for Mat and Strip Footings Supporting Seismic Class I Structures

CATEGORY I BUILDING	MAXIMUM CONTACT STRESSES BENEATH FOOTING σ u' kips/sq. ft.	BEARING MATERIAL	ULTIMATE BEARING CAPACITY OF BEARING MATERIAL $f_{bu'}$ kips/sq. ft.	$f_{bu}/\sigma u$	MAXIMUM TOTAL SETTLEMENT OF FOOTING AT σu' in.
Containment	22	Bedrock	600	27	Less than 1/10
Southeast and Southwest Auxiliary	12.3	Bedrock	600	49	Less than 1/10
Intake	26	Bedrock	600	23	Less than 1/10
Service Water Pipe Tunnel	4	In-Situ Till Deposit	40	10	Less than 1/4
Service Water Pipe Tunnel	4	Compacted Granular Backfill	50	12.5	Less than 1/4
Valve Room 1	4	Bedrock	600	150	Less than 1/10
Valve Room 2	4	In-Situ Till Deposit	40	10	Less than 1/4
Borated Water Tank	7.5	Compacted Granular Backfill	50	7	Less than 1/4
Electric Manholes 3001, 3004, 3005, 3006, 3020, 3041, and 3042	2	Compacted Granular Backfill	50	25	Less than 1/10

2.6 NON-RADIOLOGICAL ENVIRONMENTAL MONITORING

As required by the Environmental Technical Specifications, extensive non-radiological environmental monitoring programs were performed in order to determine the effects, if any, of the Davis-Besse plant on the surrounding environment.

Water quality analysis, plankton studies and benthic studies have been compared with preoperational studies to clearly demonstrate that there has been no degradation of the Lake Erie water quality or ecosystem as a result of the operation of the Davis-Besse power station. A report entitled "Environmental Impact Appraisal of the Davis-Besse Nuclear Power Station, Unit 1 on the Aquatic Ecology of Lake Erie 1973-1979" summarizes the studies which were performed (ref. 51).

Additional information regarding environmental monitoring can be found by referring to the Annual Radiological Environmental Operating Reports which have been submitted to the NRC by the Davis-Besse Nuclear Power Station (DBNPS).

The Davis-Besse plant has and continues to be subject to the monitoring requirements of the Environmental Protection Agency as delineated under the National Pollutant Discharge Elimination System (NPDES) program. Additional details regarding the NPDES program can be obtained by reviewing the NPDES permit.

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APPENDIX 2A

LAKE RESTRICTED AREAS

APPENDIX 2A - LAKE RESTRICTED AREAS

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2A.1.0 Introduction and Summary

2A.1.1 Introduction

There are areas of Lake Erie adjacent to the Davis-Besse Nuclear Power Station site which are established as Restricted Areas for use by segments of the Armed Forces in performing training missions using ground weapons. Historically, these areas were primarily used by the Erie Ordnance Depot for proof firing of artillery.

When the Erie Ordnance Depot was deactivated in 1967 and reverted to private ownership for development as the Erie Industrial Park, the main need for these restricted areas was removed but they have been maintained, with several major reductions, for limited use of certain other agencies. These include Camp Perry, an Ohio National Guard installation, and, until 1972, the Naval Air Facility in Michigan and the Lockbourne Air Force Base in Ohio. In addition to these governmental agencies, ARES Incorporated, located at the Erie Industrial Park, use the offshore restricted areas under an arrangement with the Adjutant General, State of Ohio, for test firing of ordnance.

The uses of these restricted areas were known and investigated to determine the probable effect on the Davis-Besse Station, prior to final acquisition of the station site, and were further evaluated at the time of application for and issuance of a Construction Permit to construct the station. This evaluation was included in the Preliminary Safety Analysis Report (PSAR) in 1969.

This Appendix is intended to summarize the previous reviews and to set forth the changes that have taken place which have further reduced, to great extent, the size of the areas, the usage and any potential adverse effect on the Davis-Besse Station which is considered to be negligible.

2A.1.2 Summary

The Restricted Lake Areas (Areas I and II), with their closest boundary offshore from the station site and 1-1/2 miles from the station structures, are limited to use as impact areas for small arms, artillery, and antiaircraft artillery (Figure 2A.1-2).

Use of area lying approximately ten miles north of the station (Area III), which, at the time of the Construction Permit review, was established for use by aircraft as an impact area for aerial gunnery, rocket firing, and bombing, has been discontinued and the designation of this area as a restricted danger zone area has been revoked by the U.S. Corps of Engineers (Figure 2A.1-3).

2A.1.3 Conclusions

The use of the Restricted Areas in the vicinity of the Davis-Besse Station do not affect the safety of the Davis-Besse Station and the station can be operated at this site with no undue risk to health and safety of the public resulting from this usage.

2A.1.3.1 Small Arms Firing

The small arms firing from Camp Perry is at a distance of five miles from the Davis-Besse Station and the nearest boundary of the impact area (Area I, shown on Figure 2A.1-2) is 1.8 miles. There is no possibility of small arms fire reaching the station.

2A.1.3.2 Anti-Aircraft Firing from Camp Perry

The following information is historical. Anti-aircraft training firing was suspended in 1988.

The 40 mm anti-aircraft firing area at Camp Perry is 4.5 miles from the Davis-Besse Station and the nearest boundary of the impact area (Area II, shown in Figure 2A.1-2) is 1.5 miles. The firing fans used at Camp Perry further limit the possible impact area and increases the distance of possible impact from the station.

The projectiles used carry destruct charges and fuses to prevent surface impact of intact projectiles. These destruct charges limit the maximum range of an intact projectile to approximately two-thirds of the minimum distance from the firing area to the station location.

2A.1.3.3 Ordnance Firing by ARES Incorporated

The outside firing area at the Erie Industrial Park that is used by ARES Inc. is four miles from the Davis-Besse Station and the nearest boundary of the impact area (Area II) is 1.5 miles. The firing fan for firing of all ordnance is limited to a 10° azimuthal sector, 5° east and west of north.

The limited amount of firing, type of ordnance used, type of firing, and the administrative controls associated with this firing makes the probability of a projectile impact in the immediate area of the station negligibly small.

2A.2.0 <u>Restricted Areas and Descriptions</u>

2A.2.1 <u>General</u>

Camp Perry, an Ohio National Guard training center, is located 4.5 miles southwest of the Davis-Besse site and was established in 1906. Adjacent and to the west of Camp Perry is the deactivated Erie Army Depot which is now the Erie Industrial Park. The Erie Army Depot was established in 1918 and has served also as an Ordnance Depot and as a testing center and proving grounds for artillery.

The restricted areas in Lake Erie adjacent to these installations were established to permit the firing of weapons from these installations and also for antiaircraft firing from the shore area of what is now the Davis-Besse site. Additionally, these restricted areas had been designated as impact areas for aerial bombing and strafing by aircraft from Wright Field, Dayton, Ohio, and Grosse Ile Naval Air Station in Michigan. The restricted areas established for this purpose and which were in force with local agency reduction until 1970 were as shown on Figure 2A.1-1.

In 1967, the Erie Army Depot was deactivated and the property acquired by the Ottawa County Development Corporation which in turn has leased it to Uniroyal Corporation. It is now called the Erie Industrial Park with Uniroyal as the major industry and a number of small firms. One of the early leases and occupants was the Jet & Ordnance Division of TRW, Inc., which operated an ordnance development center and used the old firing range and lake restricted areas for ordnance firing in their development and demonstration work. In 1970, TRW suspended their activities and Cadillac Gage Company assumed the lease of the range facilities for a limited weapons testing program. In 1972, ARES Inc. began operations in the Erie Industrial Park and continues to use the range facilities for weapons testing. The Cadillac Gage Company at the Erie Industrial Park no longer exists. Cadillac Gage Company suspended operation in 1995.

Since the time of deactivation of the Erie Army Depot, there have been two major modifications of the areas involved, both of which have reduced considerably the extent and usage of the areas.

2A.2.2 Surface Restricted Areas

2A.2.2.1 Authority

The Corps of Engineers, Department of the Army, U.S. Government, establishes Danger Zones and Restricted Areas within which the public can be excluded for purposes of weapon firing or other operations presenting a danger to any persons or vessels within the designated area. These Danger Zone Regulations are contained in the Code of Federal Regulations, Title 33.

2A.2.2.2 Current Regulations

The section of 33CFR pertaining to Lake Erie is Section 204.187. The current configuration of these areas and regulations governing them as published in the Federal Register, Vol. 35, No. 218, of November 7, 1970, Page 17178, and amended by notice published in the Federal Register, Vol. 37, No. 186, of September 23, 1972, Page 20026. This amendment revoked the establishment of Area III which had been used by, and was under the enforcement control of the Naval Air Facility, Detroit, at Mount Clemens, Michigan. This area (Area III) had also been used by the Lockbourne Air Force Base for aerial gunnery training.

These areas are shown on Figure 2A.1-3.

2A.2.3 Restricted Air Space

The Federal Aviation Administration has established restricted air spaces which have generally conformed to surface areas. The purpose of these restricted air spaces has been to exclude non-participating aircraft from these areas when either aerial or surface activities could be hazardous to non-participating aircraft.

These aerial restricted areas are designated and published in Part 73 of the Federal Aviation Regulations and the existing restricted air space is designated R-5502 and generally conforms to surface Areas I and II as shown on Figure 2A.1-2.

Previous to abandonment of the use of surface Area III by the Lockbourne Air Force Base operations, a restricted air space designated as R-5505 was established over surface Area III, but this restricted air space has now been eliminated. Restricted Air Space R-5505 is shown in Figure 2A.1-3 along with Restricted Air Space R-5502 as it existed between November 1972 and January 24, 1980.

2A.3.0 Usage of Restricted Areas

2A.3.1 General

The only remaining surface restricted areas are Areas I and II shown on Figure 2A.1-2. These areas have been established and are maintained under a continuing request of the Adjutant General, State of Ohio. The regulations establishing these areas also designates the Adjutant General, State of Ohio, as the Enforcing Agency. These areas are used by the Camp Perry facility.

ARES Inc. maintains and operates a test facility at the Erie Industrial Park which is principally for testing of military weapons used in conjunction with military vehicles. Part of this testing program involves limited ordnance firing into Restricted Areas I and II. This use of these restricted areas and the firing involved is under a joint use agreement with the Ohio Adjutant General through the Camp Perry Facility Manager.

2A.3.2 Camp Perry Usage

2A.3.2.1 Area I

Restricted Area I is designated as a small arms impact area and is used extensively by Ohio National Guard units for weekend and summer training for this purpose.

Camp Perry, during the late summer, is also the site of the National Rifle Matches and other small arms events sponsored by the National Rifle Association.

2A.3.2.2 Area II

The following information is historical. Anti-aircraft training firing was suspended in 1988.

Restricted Area II is used by the Ohio National Guard at Camp Perry as an impact area for antiaircraft practice firing. This usage is approximately one week in the summer.

The antiaircraft weapons are 40 mm twin-mount units located at the northwest corner of the Camp Perry reservation, 4.5 miles from the Davis-Besse Station. The targets are towed by radio-controlled planes launched from Camp Perry.

All artillery firing at Camp Perry is controlled by very close supervision and the azimuth is controlled on all guns by the use of limiting stakes which prevent any gun firing outside of the authorized firing fans. The minimum distance from the station to the 40 mm firing fan is approximately 2.5 miles.

In addition to the limiting stakes on the gun housings, the gun azimuths are closely checked and all firing is under the direction of a safety officer or NCO stationed on the deck of each gun. All firing procedures are governed by the safety regulations.

The ammunition used by the 40 mm antiaircraft weapons are conventional high explosive tracer rounds. The explosive charge is 63 grams of TNT equivalent. These high explosive rounds are equipped with a self-destructing fuse that functions when the tracer is expended. This effectively limits the range of the round to a maximum of 5,200 yards and ensures that no intact high explosive projectiles will fall to the ground. As the station location is over 10,000 yards from the firing location, over 1.5 miles outside the restricted area and 2.5 miles outside the limits of the firing fan, and at an angle of approximately 38° to the west of the westerly limit of the firing fan, the probability of a 40 mm projectile impacting the reactor area is vanishingly small. Under these conditions, the hazards to the Davis-Besse Station as a result of the 40 mm antiaircraft firing operations must be considered negligible.

2A.3.3 ARES Inc. Usage

ARES Inc., located at the Erie Industrial Park, maintains a test range for military weapons. This testing program involves firing of ordnance including automatic weapons, mortars, and cannons. ARES Inc. conducts ordnance firing approximately two days per week throughout the year.

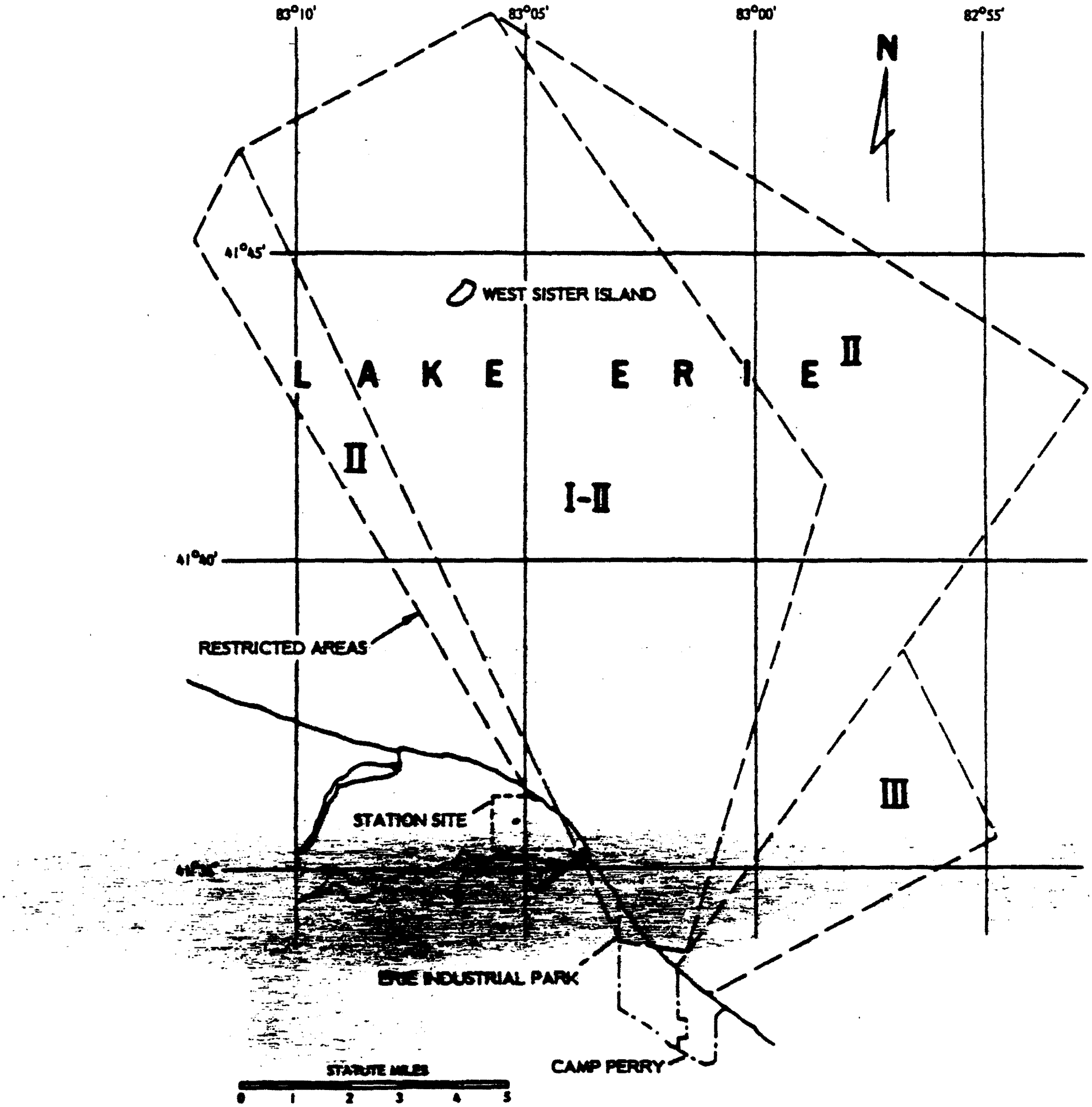
Test-range firing is limited to an azimuth of 5° east and west of north. Firing is conducted from a fixed gun mount or from parked, stationary vehicles. Most of the firing is done from a zero or slight elevation angle which causes projectile impact in the marsh area in front of the test-firing range. Some firing is done with higher elevations which causes impact in the lake area, mostly Area I with very limited impact in Area II.

During the reporting period from October 1970 through September 1971, Cadillac Gage company conducted ordnance firing on 19 days while, during the reporting period of October 1971 through September 1972, this usage had decreased to a total of eight days. The Cadillac Gage Company suspended operations in 1995.

2A.4.0 Future Activities

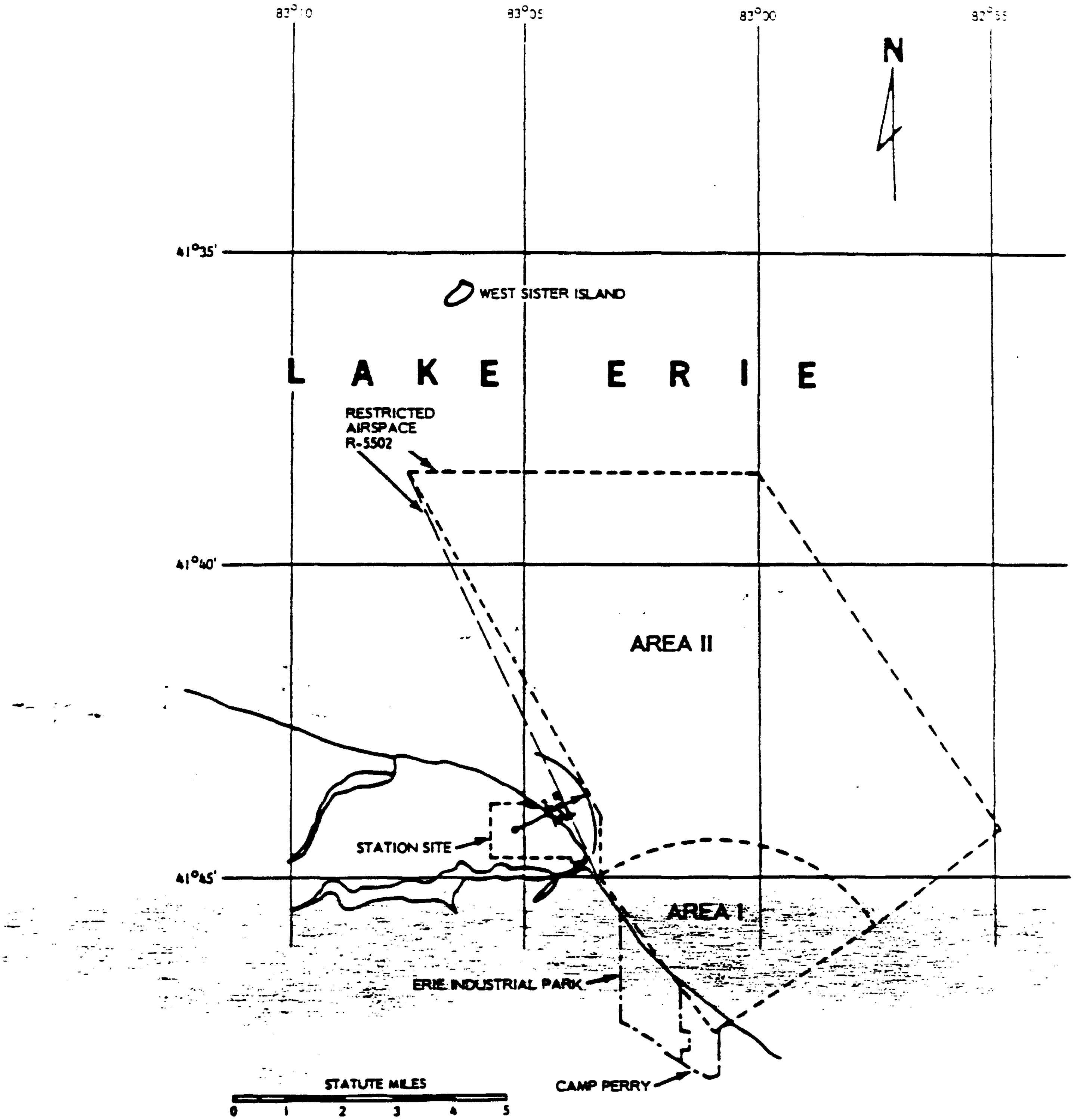
The revocation of Area III has reduced the restricted space and configuration to an arrangement that would prohibit the use of the restricted areas for aircraft operations similar to that previously conducted in Area II and as such, eliminates any potential of aircraft impacts which might have previously been present.

The size and configuration of Areas I and II limits their use to type of usage presently being practiced. This type of usage is very limited and is not expected to show any increase. This usage, due to its limited extent, type of usage, and controls involved, presents no potential hazard to the Davis-Besse Station.



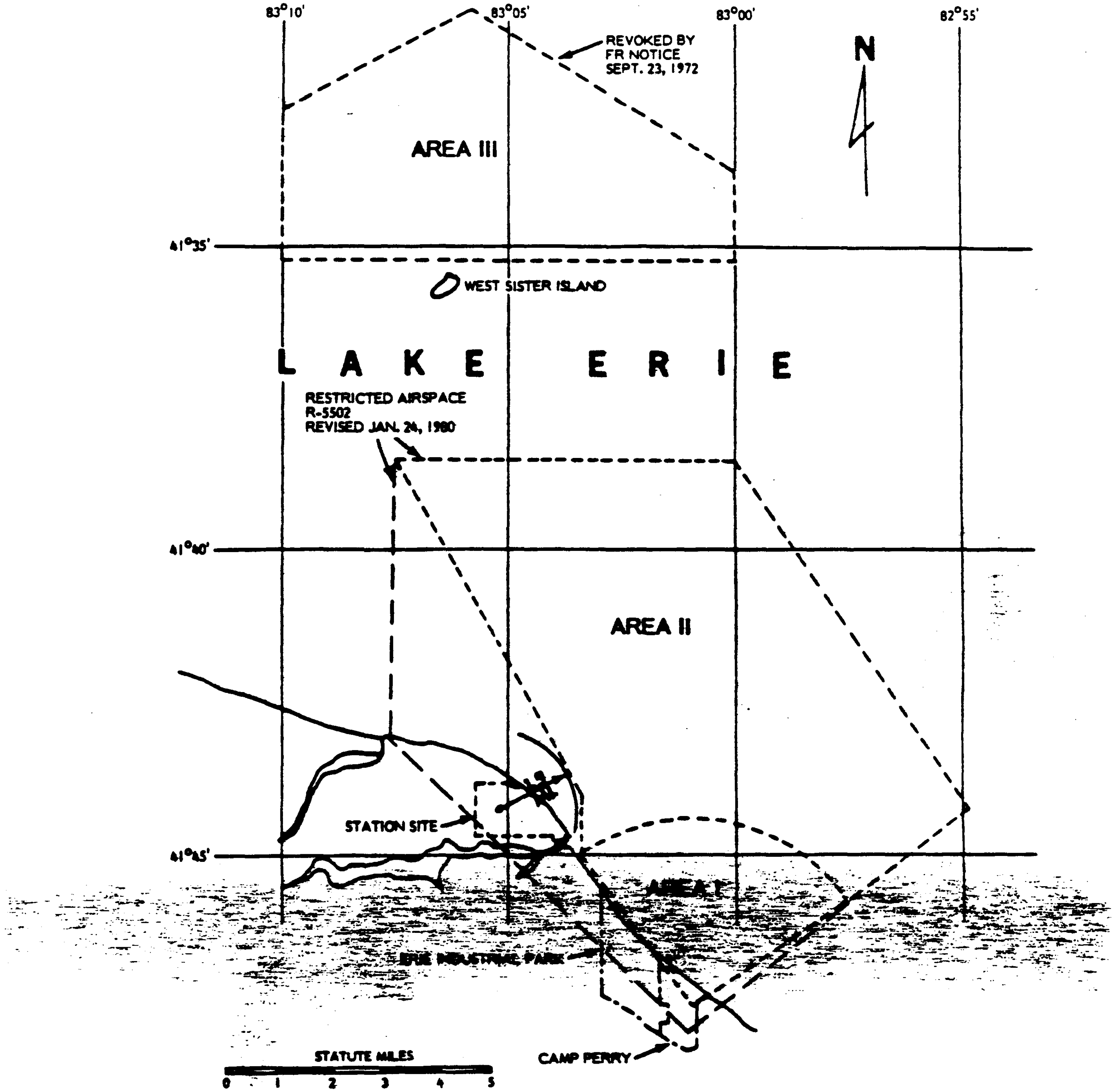
DAVIS-BESSE NUCLEAR POWER STATION
RESTRICTED AREA BOUNDARIES
JANUARY 1, 1967
FIGURE 2A.1-1

REVISION 1
JULY 1983



DAVIS-BESSE NUCLEAR POWER STATION
RESTRICTED AREA BOUNDARIES
JANUARY 24, 1980
FIGURE 2A.1-2

REVISION 1
JULY 1983



DAVIS-BESSE NUCLEAR POWER STATION
RESTRICTED AREA BOUNDARIES
NOVEMBER 1, 1972
FIGURE 2A.1-3

REVISION 1
JULY 1983

APPENDIX 2B

AQUATIC BIOLOGY OF LAKE ERIE

IN VICINITY OF LOCUST POINT, OHIO

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OCTOBER, 1972

FOR

THE TOLEDO EDISON COMPANY

APPENDIX 2B

AQUATIC BIOLOGY OF LAKE ERIE IN THE VICINITY OF LOCUST POINT, OHIO

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2B.1.0 Introduction

The aquatic biology of the Locust Point area of Lake Erie has received considerable attention in the past several years because of the announcement and eventual construction of the Davis-Besse Nuclear Power Station adjacent to the lake on Locust Point. Recent biological investigations by the Ohio Department of Natural Resources, The Ohio State University and The University of Michigan have been conducted or are underway near the plant site. The purpose of this report is to summarize and synthesize the results of these studies and thereby provide a baseline assessment of the aquatic organisms and their ecological relationships prior to the operation of the nuclear plant.

The following section of this report is included as a review of the recent research projects dealing with the aquatic biology of Lake Erie in the vicinity of Locust Point. Project titles, authors, supporting agencies, objectives, methods and publications are listed for each study.

2B.2.0 Research Projects

2B.2.1 Ohio Department of Natural Resources

Project:
Physical Characteristics of the Reef Area of Western Lake Erie

Principal Investigators:

Dr. Charles E. Herdendorf
Geologist and Lake Erie Section Head,
Division of Geological Survey
Ohio Department of Natural Resources

Mr. Lawrence L. Braidech
Geologist, Division of Geological Survey
Ohio Department of Natural Resources

Supporting Agencies:

Research on this project (AFCS-1) was supported by the U.S. Department of Interior, Bureaus of Commercial and Sport Fisheries through the anadromous Fish Act in cooperation with Ohio Department of Natural Resources, Division of Wildlife. The Division of Geological Survey provided 50% of the cost of this research.

Objectives:

A three-year (April 1967-April 1970) study was conducted to determine the physical characteristics of the reefs and surrounding areas in western Lake Erie. The investigation was undertaken to provide State and Federal fisheries biologists with information in support of their resource management programs, particularly as the physical makeup of the area relates to the spawning, nursery, and feeding grounds for such species as walleye, white bass and channel catfish. The objectives of the project were to map the area bathymetrically and by sediment and bedrock type and to measure the movements and physicochemical properties of the overlying lake water. This information was used to determine the chronology of ecological and geological events in this portion of the lake and to relate the existing physical environment to the equilibrium of the ecosystem.

Research Methods:

Thirteen reefs and shoals were mapped in detail using a recording echo sounder. Surface sediment samples were taken at half-mile intervals in a grid pattern resulting in 1,383 stations; the sampler was a 1.5 liter-capacity La Fond-Deitz. Sediment cores were taken at 75 stations using the hydraulic jetting method. This technique permitted probing to the bedrock surface at most locations. The process of contemporary sedimentation was studied by collecting sediment as it was being deposited on six reefs with a collection device designed for this study. Sediment samples were analyzed for mineral content and grain size. Selected samples were analyzed for organic content and benthic organisms. Currents and several other physicochemical water properties (including temperature, conductivity, dissolved oxygen, transparency, pH, turbidity and alkalinity) were measured at 68 stations.

Publications and Reports:

Herdendorf, C. E. 1968. Sedimentation studies in the south shore reef area of western Lake Erie. Proc. 11th Conf. Great Lakes Research, Milwaukee, p. 188-205.

Herdendorf, C. E. 1970. Limnological investigations of the spawning reefs of western Lake Erie with particular attention to their physical characteristics. Ph.D. Diss. Ohio State Univ. 203 p.

Herdendorf, C. E. and L. L. Braidech. 1970. A study of the major reef areas in the western basin of Lake Erie. Ohio Dept. Natural Resources, Div. Geological Survey, Anadromous Fish Proj. AFCS-1, Final Rept. 139 p.

Herdendorf C. E. and L. L. Braidech. 1972. Physical characteristics of the reef area of western Lake Erie. Ohio Dept. Natural Resources, Div. Geological Survey Rept. Invest. 82. 90 p.

2B.2.2 Ohio State University

Project:
Environmental Evaluation of a Nuclear Power Plant on Lake Erie.

Principal Investigators:

Dr. Richard A. Tubb
Leader, Ohio Cooperative Fishery Unit
Ohio State University

Dr. Loren S. Putnam
Director, F. T. Stone Laboratory
Ohio State University

Dr. Charles E. Herdendorf
Director, Center for Lake Erie Area Research
Ohio State University

Dr. Elizabeth H. Hair
Research Associate, Center for Lake Erie Area Research
Ohio State University

Supporting Agencies:

Research on this project (F-41-R) is being supported by the U.S. Department of Interior, U.S. Fish and Wildlife Service through the Federal Aid in Sport Fish Restoration Act (Dingell-Johnson Act) in cooperation with the Ohio Department of Natural Resources, Division of Wildlife. The Ohio State University provides 25% of the cost of this research.

Objectives:

In June 1969 research was begun on a project designed to provide baseline estimates of benthic, planktonic, and fishery populations before and after thermal discharge to Lake Erie from the Davis-Besse Nuclear Power Station. This project is scheduled to continue through the initial operating years of the plant.

Research Methods:

Three transects in the Locust Point area were used in 1969, 1970 and 1971 for sampling fish populations. Transect 1 begins 1,000 ft. west of the Toussaint River and runs toward West Sister Island. Transect 2 runs from the mouth of the Toussaint River to Niagara Reef and Transect 3 starts 1,000 ft. east of the river and runs toward South Bass Island. Commercial fisherman were contracted to operate their 2750 ft. seines in the area of Transect 3 from 1969 to 1971 and in the area of Transect 1 in 1972. Fish were sampled with this gear once a month from June through October, 1969, May through October, 1970, April and May, 1971, and July to September 1972. A 150-ft. bag seine was fished at five locations; one haul was made at the base of each transect, one haul between Transects 1 and 2 and one haul between Transects 2 and 3. The net was 6 ft. deep with 3/4" mesh in the wings and 1/4" mesh in the bag. Fish were sampled by this method in July, August and October, 1969, May through October, 1970 and May 1971. July to September, 1972, the same method was used at three locations near the proposed water intake and discharge pipelines. Three experimental gill nets were set, one along each transect in 1969 and 1970. They were fished overnight perpendicular to shore with the small mesh set closest to shore in about 8 feet of water. The nets were 6 ft. deep and 150 ft. long, consisting of contiguous 25-ft. panels of stretch mesh from ¾" to 5". Gill nets were set once a month from August through October, 1969 and in May and August, 1970. In 1972, this method was used in August at two locations; the proposed intake structure (3000 ft. offshore) and the proposed discharge structure (1500 ft. offshore).

The catch of each species by each method was determined after every sampling period, as was the total catch for each gear. Except for carp and sheepshead all fish were weighed and measured and scales taken from representatives of each size group. When the commercial seine catch contained too many carp and sheepshead to process feasibly, the fishermen estimated the total catch. Catch per unit effort (CPE) was determined for each gear and a comparison was made between the relative numbers of individuals and species for each gill net and 150-ft. seine haul.

Representatives of different size groups and species were weighed, measured and their stomachs preserved in 5% formalin solution as soon as possible after capture. Only live, seined fish were used for this study. Stomach contents were identified as far as possible and enumerated. Because of the small stomach volumes and diversity of food items, the numerical approach was used rather than the volumetric method. Frequency of occurrence was calculated for each food item for each fish species. Differences from month to month were examined and food habits of different sized individuals of the same species were compared.

Scales were read using a microprojector and annular measurements made to enable back calculation of growth rate. Length-frequency plots were used for determining ages of shiners.

Plankton samples were taken at 23 stations along three transects radiating outward from the power plant. A 3-liter Kemmerer water bottle was used to collect samples at 1 and 3 meters. Samples were concentrated by passage through a No. 20 bolting cloth and zooplankters were then fixed in 5% formalin. The smaller nannoplankters were lost with this procedure, but the larger zooplankters fed upon by fishes were retained and an overall estimate of the zooplankters was readily accomplished using this procedure. Zooplankters were counted in a Sedgewick-Rafter counting chamber at 100X magnification. Individual species were identified when necessary at 980X magnification.

Twenty one stations in the Locust Point area were established in 1969 for sampling benthos populations. These varied in depth from 6 to 22 feet and included substrates of silt, sand, detritus, gravel, clay, small rocks, boulders and bedrock. The same 21 stations were sampled monthly from June through October, 1969, and from May through October, 1970. Late in 1970, the Toledo Edison Company announced plans to build a cooling tower substantially reducing the area affected by the heated effluent. The sampling program for the spring of 1971 was modified accordingly by deleting the deeper, offshore stations and adding stations at the 6 to 10-foot contours.

Samples were collected in 1969 with a Petersen dredge ($A=0.0833m^2$). In 1970, a pump-type sampler was used on the boulder and bedrock reef areas. This sampler consisted of a gasoline-powered centrifugal pump, 25 feet of 2-inch pressure hose and a 12-inch diameter head ($A=0.073m^2$). This device was operated in cooperation with the Ohio Division of Wildlife, Lake Erie Research Unit. During 1969, each sample consisted of three dredge hauls which were analyzed separately. However, the replicates proved to be very similar, and in 1970 and 1971 only one dredge haul was taken at each station. All samples were sieved through a Number 35 U.S. Soil Series screen (0.5 mm mesh) and preserved in 10% formalin. Samples taken in 1970 and 1971 were stained with Rose Bengal. Samples were rough-sorted using a 10X dissecting microscope and the organisms identified to genus (species when possible) and counted.

In 1972, emphasis was shifted from offshore to near-shore studies of the aquatic biology of the Locust Point area because of the addition of a cooling tower and concern over environmental disruption which might be caused by the construction of a barge channel and intake-discharge pipelines in the lake. Detailed soundings of the near-shore zone were made with a recording fathometer and sediment samples were taken at 38 stations within 4000 feet of the shore. Benthos and plankton samples were taken at 25 of these stations, 15 of which were within 1000 feet of the shore. Scuba diving surveys of the lake bottom within 1000 feet of the shoreline were made to determine the extent of mollusk populations. Water chemistry and physical parameters, including currents, were studied at six stations.

Publications and Reports:

Tubb, R. A. 1972. Environmental evaluation of a nuclear power plant. Fish, plankton and benthos populations prior to discharge. Ohio State Univ. Project F-41-R-3 Completion Rept. 57 p.

Herdendorf, C. E. and E. N. Hair. 1972. Environmental evaluation of a nuclear power plant of Lake Erie. Fish, plankton and benthos populations prior to discharge. Ohio State Univ. Project F-41-R-4 Monthly and Quarterly Progress Repts., June-September 1972. 45 p.

2B.2.3 University of Michigan

Project:

Hydrological Surveys for the Locust Point Power Plant

Principal Investigators:

Dr. John C. Ayers
Professor, Great Lakes Research Division
University of Michigan

Dr. Dean E. Arnold
Research Limnologist, Great Lakes Research Division
University of Michigan

Dr. Robert F. Anderson
Research Associate, Great Lakes Research Division
University of Michigan

Dr. Norbert W. O'Hare
Research Associate, Great Lakes Research Division
University of Michigan

Supporting Agencies:

The University of Michigan, Great Lakes Research Division conducted this research under a contract with The Toledo Edison Company, Toledo, Ohio.

Objectives:

In 1969 studies were conducted to determine the phytoplankton, zooplankton, benthos and fish populations of Lake Erie in the vicinity of Locust Point. This information was used to prepare a statement on the anticipated thermal effects of Davis-Besse Nuclear Power Station.

Research Methods:

Phytoplankton, zooplankton, and benthos collections were made at 11 stations in May 1969, and at 20 stations in October 1969. Details of sampling techniques were not given in reports of this research. Data on fish were not collected directly, but obtained from various government reports and interviews with fisheries' biologists working in the area.

Publications and Reports:

Ayers, J. C., R. F. Anderson, N. W. O'Hare, D. E. Arnold, and C. C. Kidd. 1970. Hydrological surveys for the Locust Point power plant. Part III. Preliminary biological, fisheries, and radiological studies. Part IV. The 1969 phytoplankton collections in the Locust Point region, Univ. Michigan, Great Lakes Research Division Spec. Rept. 45. 104 p.

2B.3.0 Habitat

Locust Point is a gently curving headland on the south shore of western Lake Erie, approximately 10 miles west of Port Clinton (Figure 2B.3-1). The point has a relatively stable barrier beach which separates Navarre marsh from the lake. The shore is apparently not tending to straighten itself or advance inland over the marsh which is usual for barrier beaches with such a configuration.

Locust Point is at a position of diverging littoral (alongshore) drifts of sand which ordinarily would result in the beach being starved of sand because of movement east and west away from the headlands. However, the shore is apparently maintained at near equilibrium by replenishment from an extensive sand and gravel deposit which lies north of a narrow strip of compact glaciolacustrine clay that fronts the Locust Point area beyond the sandy near shore zone (Figure 2B.3-2). Transportation of this material from offshore to the beach can be accomplished by two forces: wave action and ice shove. Most of the sand probably migrates shoreward by wave action and currents generated by northwest and northeast storms. Evidence for the shoreward movement of sand can be found in the position of bars before and after major storms. For example, fathometer profiles before (13 June 1972) and after (28 June 1972) tropical storm Agnes revealed that the two offshore sand bars at Locust Point migrated 20 to 25 feet shoreward as a result of the wave attack from the northwest.

Hydrographic surveys show a very gentle slope of the lake bottom from the shore out for a distance of at least 4000 feet (Figure 2B.3-1). Two sand bars lay in the near shore zone, one at 120 feet offshore, the other at 280 feet from the beach. The deeper area between the beach and the first sand bar has a thin bottom layer of fluffy silt and broken shells over the sand. The inshore slope of the first bar contains an abundant population of naiad clams. The sand bottom, generally medium- to fine-grained, extends to 800 feet offshore (5.0 feet water depth, IGLD, 1955). At this point the bottom deepens by 0.5 foot and is composed of hard, glaciolacustrine clay which is 500-700 feet wide. Lakeward the bottom again becomes sandy and the sand increases in thickness in a lakeward direction. The lake reaches a depth of 10 feet at distances of 2200 feet offshore and 12 feet at 4000 feet offshore. The sand and gravel bottom persists lakeward to rocky reefs three miles offshore (Figure 2B.3-1).

The offshore reefs consist of bedrock and associated rock rubble and gravel. The topography of the reef tops ranges from rugged surfaces caused by bedrock pinnacles and large boulders, to smooth slabs of horizontally bedded rock. In places the exposed bedrock has the appearance of low stairs with the steps dipping slightly to the east from the crest to the fringe of the reef. All of the bedrock formations that form the reefs are carbonate rocks with abundant solution cavities, in many cases up to 1 or 2 cm in diameter. The bedrock itself is commonly masked by rubble composed of both autochthonous and glacial origin and ranging from small pebbles to boulders up to 5 feet in diameter. On the reefs, isolated patches of sand and gravel fill vertical joint cracks and small depressions in the bedrock; at the fringes the reefs, sand and gravel beds or glacial till lap over the rock. During quiet periods the rocks are often covered by a thin layer of fluff, organic-rich silt which can be several millimeters thick (Herdendorf, 1970).

Lakeward of the reefs the depths increase rapidly to 24 feet. Here the bottom is composed of mud (semi-fluid silt and clay-sized particles) and less than 10% sand (Figure 2B.3-2).

2B.4.0 Plankton Populations

Phytoplankton Population:

The only detailed report of the phytoplankton population in the Locust Point area is that of Ayers (1970) for eleven stations sampled in May, 1969. The mean number of cells per liter for the eleven stations was 4,459,594. The phytoplankton density ranged from 1,728,624 cells per liter to 11,953,521 cells per liter.

Ayers identified 31 species of phytoplankton from the May, 1969 samples (Table 2B.4-1). Tubb (1972) reported five additional species (four of which were blue-greens) which were not found in Ayers' mid-May samples. Tubb did not quantify phytoplankton in his investigation.

Zooplankton Population:

Ayers (1970) and Tubb (1972) have reported 25 zooplankton taxa from the Locust Point area (Table 2B.4-1). Tubb (1972) found the most frequently occurring zooplankton to be copepod nauplii, found in 100% of samples; cyclopoid and calanoid copepods (in 97.2 and 84.9% of the samples), the rotifers Keratella cochlearis and Polyarthra spp. (95.2% and 93.7%) and the cladocerans, Daphnia l retrocurva and Bosmina spp. (80.1% and 86.3%).

Seasonal changes in mean zooplankton density during the ice-free months are shown in Figure 2B.4-1. Peak densities appeared to be reached in mid-July to August, with spring and fall densities being lower. Copepods and Daphnia spp. reached their highest density in late June and early July, while Bosmina spp. was most abundant in August and September.

Tubb (1972) found that neither water depth nor station location (nearshore or offshore) affected zooplankton density or diversity; however Ayers (1970) reported lower densities at his nearshore stations.

Ayers reported numbers of organism for May and October, 1969 which were significantly lower than any reported by Tubb. This may have been due to differences in sampling or counting techniques rather than actual population differences.

Tubb reported a range of population densities from 13 organisms/l at one station in April, 1971 to 3902 organism/l at one station in July, 1970. Ayers summed the number of organisms present at each station in both May and October, 1969 and reported a range of densities from 37.5 organisms/l to 204.8 organisms/l.

TABLE 2B.4-1

Aquatic Organisms Found in the Locust Point Area of Lake Erie

1969-1972

PHYTOPLANKTON

Diatoms
- Diatoma tenuis v. elongata[1]
- Melosira binderana[1]
- M. granulata[1]
- M. sp[2]
- Synedra ulna[1]
- S. acus[1]
- Fragilaria intermedia[1]
- F. capucina[1]
- F. crotonensis[1]
- F. sp.[2]
- Asterionella formosa[1]
- A. sp.[2]
- Cyclotella spp.[1]
- Novicula spp.[1]
- Tabellaria fenestrate[1]
- T. sp.[2]
- Surirella spp. [1]
- Nitzchia spp. [1]
- Stephanodiscus spp.[1]
- Cymbella spp. [1]
- Gomphonema sp. [1]

Green Algae
- Clothrix spp.[1]
- Pediastrum dulex[1]
- P. sp.[2]
- Staurastrum sp. [2]
- Scenedesmus abundans
- S. quadricauda[1]
- Dictyosphaerium pulchellum[1]
- Ankistrodesmus spp.[1]
- A. falcatus[1]
- Micractinium pusillium[1]
- Oocystis solitaria[1]
- Lagerheimia longiseta[1]
- Golenkinia radiata[1]
- Actinastrum hantzschii[1]
- Closteriopsis longissima[1]
- Ceratium spp.[2]

PHYTOPLANKTON (cont.)

Blue Green Algae
- Oscillatoria spp.[1]
- Microcystis sp.[2]
- Anabaena sp.[2]
- Aphanizomenon sp. [2]

ZOOPLANKTON

Rotifera
- Asplanchna sp.[1,2]
- Brachionus sp.[2]
- Filinia sp.[2]
- Keratella cochlearis[2]
- K. quadrula[2]
- Kellicottia sp. [2]
- Philodina sp.[2]
- Polyarthra sp. [2]
- Trichocerca sp. [2]

Calanoid Copepoda[2]
- Diaptomus sp.[1]
- Eurytemora affinis[1]

Cyclopoid Copepoda[1,2]

Cladocera
- Bosmina sp.[1,2]
- Chydorus sp.[2]
- Chydorus sphaericus[1]
- Ceriodaphnia sp. [1,2]
- Daphnia galeata[2]
- D. parvula[2]
- D. retrocurva[1,2]
- D. spp.[1]
- Diaphanosoma sp.[2]
- Leptodora kindtii[1,2]
- Sida crystalline[1]
- Simocephalus sp. [2]

Onstracoda[1]

BENTHOS

Coelenterata
- Hydra sp.[2]

Platyhelminthes
- Turbellaria
- Planariidae[2]

Annelida
- Hirudinea[1,2]

Oligochaeta
- Limnodrilus hoffmeisteri[2]
- L. maumeensis[2]
- L. cervix[2]
- L. claparedeanus[2]
- L. claparedeanus cervix[2]
- L. udekemianus[2]
- Aulodrilus sp.[2]
- Peloscolex ferox[2]
- Potamothrix moldaviensis[2]
- P. vejdovskyi[2]
- Brachyura sowerbyi[2]
- Nais sp.[2]
- Stylaria sp.[2]

Bryozoa[2]

Isonoda[2]
- Asellus sp.[2]

Amphipoda[1]
- Gammarus sp.[2]
- Hyalella azteca[2]

Decapoda[2]
- Orconectes sp. [2]

Ephemeroptera[2]
- Caenis sp. [2]

Trichoptera[2]
- Oecetis sp.[2]
- Athripsodes sp.[2]
- Polycentropus sp. [2]

Chironomidae[2]
- Chironomus (Chironomus) sp. [2]
- C. (Cryptochironomus) sp.[2]

TABLE 2B.4-1 (Continued)

Aquatic Organisms Found in the Locust Point Area of Lake Erie

1969-1972

BENTHOS (cont.)

Chironomidae[1]
- Polypedilum sp.[2]
- Pseudochironomus sp. [2]
- Tanytarsus sp.[2]
- Procladius sp.[2]
- Coelotanypus sp.[2]
- Cricotopus sp.[2]
- Psectrocladius sp.[2]

Gastropoda[1]
- Amnicola sp.[2]
- Bithynia sp.[2]
- Physa sp.[2]
- Pleurocera-Goniobasis[2] [4]
- Gyraulus sp.[2]
- Valvata sp.[2]

Sphaeriidae[1]
- Sphaerium sp.[2]
- Pisidium sp.[2]

Unionidae
- Quadrula pustulosa[3]
- Amblema plicata[3]
- Leptodea fragilis[3]
- Potamilus alatus[3]
- Ligumia recta[3]
- Lamsilis ventricosa[3]
- L. radiata[3]

PERIPHYTON

Green Algae
- Cladophora sp.[3]

FISH

Lepisosteidae[2]
- Lepisosteus osseus[2]

Amiidae[2]
- Amia calva[2]

Clupeidae[2]
- Alosa pseudoharenqus[2]
- Dorosoma cepedianum[2]

Salmonidae[2]
- Oncorhynchus kisutch[2]

Osmeridae[2]
- Osmerus eperlanus mordax[2]

Esocidae[2]
- Esox lucius[2]

Catostomidae[2]
- Carpiodes cyprinus[2]
- Moxostoma erythrurum[2]
- Catostomus commersoni[2]
- Minytrena melanops[2]

Cyprinidae[2]
- Cyprinus carpio[2]
- Carassius auratus[2]

FISH (cont.)

Cyprinidae[2]
- Notropis atherinoides[2]
- Notropis spilopterus[2]
- Notropis hudsonius[2]
- Hybopsis storeriana[2]

Ictaluridae[2]
- Ictalurus nebulosus[2]
- Ictalurus punctatus[2]
- Noturus flavus[2]

Serranidae[2]
- Morone chrysops[2]

Centrarchidae[2]
- Pomoxis nigromaculatus[2]
- Pomoxis annularis[2]
- Ambloplites rupestris[2]
- Micropterus d. dolomieui[2]
- Micropterus s. salmoides[2]
- Lepomis cyanellus[2]
- Lepomis humilis[2]

Percidae[2]
- Perca flavescens[2]
- Stizostedion v. vitreum[2]
- Percina caprodes[2]

Sciaenidae[2]
- Aplodinotus grunniens[2]

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[1] Ayers (1970)
[2] Tubb (1972)
[3] Herdendorf and Hair (1972)
[4] Cannot distinguish genera with certainty.

2B.5.0 Benthos Populations

Table 2B.4-1 shows the forty-nine taxa of benthic macroinvertebrates which have been reported from the Locust Point area by Ayers (1970) and Tubb (1972). Of these, 13 were species of Oligochaeta, 9 were Chironomidae, 9 were Pelecypoda and 6 were Gastropoda. Species diversity did not appear to change greatly over the seasons.

During the ice-free months, there was a gradual increase in the mean number of organisms/m² (Figure 2B.4-2). This was primarily due to the addition of immature oligochaetes to the community in late summer. The apparent increase Tubb (1972) found in number of organisms/m² from 1969 to 1970 was probably the result of an improvement in the sorting and counting technique in 1970.

Both Ayers (1970) and Tubb (1972) noted smaller populations at their nearshore stations (6 ft. depth and 300 yds. offshore) than at stations farther offshore. In addition, Tubb found the greatest population density to be at stations between 10 and 15 feet deep while the greatest species diversity was at stations between 10 and 12 feet deep.

Tubb also observed that substrate type affected species distribution. He found the greatest species diversity in gravel-small rock substrate and the greatest population density (number of organisms/rn²) in silt-detritus substrate. Clay-gravel substrate supported the fewest organisms and the least diversity; this substrate material is till which was deposited by glacial action.

2B.6.0 Fish Populations

Commercial Fish Production:

In the vicinity of Locust Point, commercial shore seine fishermen catch over 170,000 pounds of fish annually. During the period 1963-1972, eleven species were taken commercially. Of these species, 69.4% were carp and goldfish, 22.9% were catfish, and 6.8% were white bass. The remaining 0.9% consisted largely of perch, walleye, sheepshead, bullheads, and dogfish. Peak production occurred in the spring, usually in May. Late summer catches were the lowest followed by a moderate increase in the fall.

Table 2B.6-1 documents ten years of production by the two prominent commercial fishing operations in the Locust Point area: Clifford Biggert (Oak Harbor) and Virgil St. Clair & Sons (Oak Harbor). Seines used by these operators are generally 160 rods to 170 rods long and contain bags with 2 1/4 to 2 7/8 inch mesh (stretch). Nearly 600 seine hauls were made during this period with an average production of approximately 2900 pounds per unit effort.

Natural populations:

Information concerning fish populations in the immediate Locust Point area comes primarily from Tubb (1972). In addition, Ayers' (1970) investigation included data from a U. S. Bureau of Commercial Fisheries trawling station at Bono, about 10 miles NW of Locust Point and the Ohio Division of Wildlife has some trawling and gill netting stations in the reef area (Niagara Reef, West Reef) and has for the past 10 years studied Walleye egg deposition on Toussaint, Crib, and Niagara Reefs (Baker, 1969).

Tubb found 33 fish species in the Locust Point area (Tables 2B.4-1 and 2B.6-2), although the greatest number taken in any one month was 21 (May, 1970). Freshwater drum, carp (including goldfish and/or hybrids), gizzard shad and white bass were taken during every month sampled.

Table 2B.6-3 lists the common and scientific names for the fish species found in the Locust Point area. Tables 2B.6-4, 2B.6-5, and 2B.6-6 show total catch per unit effort for each sampling method and the relative abundance (5) of each species taken. Carp, gizzard shad and drum were the most abundant species; in 7 of the 12 months sampled, carp made up more than 60% of the catch. Total catches were greatest in the late spring and early fall and lowest in mid to late summer.

Tubb combined data from several sources to give a view of the age structure of fishes in the Locust Point area. Table 2B.6-7 shows the mean length and the length range, where available, for each age class of most of the frequently-collected species. Tubb found that early spring samples were comprised mostly of adult individuals. As summer progressed young-of-the-year individuals, as well as yearlings, became more abundant in the samples.

Table 2B.6-8 summarizes the food habits of some Locust Point area fishes. From June 1969 through May 1971, 1919 stomachs were examined, 71.8% of which contained food. A seasonal pattern was evident, with a greater percentage of stomachs containing food in the summer months than in the spring and fall.

All species of fish utilized food organisms found in the Locust Point area, both in the open lake and along the shore or in the river. Food items included benthic invertebrates from all substrates in the area (silt and sand, gravel and clay, and reef). Zooplankters utilized corresponded with those predominant in most of the plankton samples.

The wide variety of food items seemed to indicate relatively free movement into and out of the Toussaint River and the marsh area by all fish species except perch, which seemed to feed exclusively on open-lake species.

TABLE 2B.6-1

Lake Erie Commercial Fish Production
In the Vicinity of Locust Point

1963-1972

1963

Species	April	May	June	July	Aug.	Totals
	Pounds					
Buffalo	20					20
Bullheads	120					120
Carp	15,176	14,417	14,810	15,175	15,350	74,928
Catfish	2,508	10,010	9,214	1,093		22,825
Perch	855					855
Sheepshead	5					5
Walleye	25	12				37
White bass	50	1,433	1,075			2,558
White sucker	15					15
Totals	18,774	25,872	25,099	16,268	15,350	101,363

1964

Species	March	April	May	June	July	Totals
	Pounds					
Buffalo	100					100
Bullheads	29	42				71
Carp	10,560					10,560
Catfish		5,800	19,348	2,646	1,239	29,033
Perch	10	37				47
Sheepshead	25	151				176
Walleye	47	149	3			199
White bass	9	1,529	7,518	4,407	1,000	14,463
White sucker	70					70
Totals	10,850	7,708	26,869	7,053	2,239	54,719

1965

Species	April	May	June	July	Aug	Totals
	Pounds					
Buffalo	17	20				37
Bullheads	24					24
Carp	16,050	11,490	1,200	20,925		49,665
Catfish	1,981	27,165	11,339	1,200	100	41,785
Perch	304					304
Quillback	12					12
Sheepshead	466					466
Walleye	270	38				308
White bass	666	5,968	3,591			10,225
White sucker	30					30
Totals	19,820	44,681	16,130	22,125	100	102,856

TABLE 2B.6-1 (Continued)

Lake Erie Commercial Fish Production
In The Vicinity of Locust Point

1963-1972

1966

Pounds								
Species	March	April	May	June	July	Aug.	Sept.	Totals
Carp	3,000	102,500	10,579	8,000	15,060	37,230	3,100	179,469
Catfish		1,120	14,307	31,855	2,215	1,335		50,832
Perch	25	85						110
Sheepshead		1,200						1,200
White bass	10	520	6,858	12,169	200			19,757
Totals	3,035	105,425	31,744	52,024	17,475	38,565	3,100	251,368

1967

Pounds							
Species	April	May	June	July	Aug.	Sept.	Totals
Bullheads	45						45
Carp	8,400	700	32,490	9,270	12,700	14,460	78,020
Catfish	3,725	4,354	16,641	1,771	487		26,978
Perch	50						50
Walleye	135						135
White bass	273	1,343	1,773		179		3,568
Totals	12,628	6,397	50,904	11,041	13,366	14,460	108,796

1968

Pounds							
Species	March	April	May	June	July	Aug.	Totals
Buffalo		25		63	272		360
Carp	12,225	27,600	33,000	24,127	25,535	28,519	151,006
Catfish		13,097	8,875	19,594	2,265	308	44,139
Perch	100	205	5				310
Sheepshead		41					41
Walleye	20	504	93				617
White bass		2,414	3,219	10,413	113		16,159
White sucker		9					9
Totals	12,345	43,895	45,192	54,197	28,185	28,827	212,641

TABLE 2B.6-1 (Continued)

Lake Erie Commercial Fish Production
In The Vicinity of Locust Point

1963-1972

1969

Pounds							
Species	March	April	May	June	July	Aug.	Totals
Bullheads		35					35
Carp	7,000	50,349	6,500	21,500	12,000	6,000	103,349
Catfish		25,653	27,271	6,182	11,917	320	71,343
Perch		1,586					1,586
Quillback		70					70
Sheepshead		300					300
Walleye	30	1,156	75				1,261
White bass		5,730	6,670	2,080	7,725	155	22,360
White sucker		300					300
Totals	7,030	85,179	40,516	29,762	31,642	6,475	200,604

1970

Pounds									
Species	March	April	May	June	July	Aug.	Sept.	Oct.	Totals
Carp	17,900	44,295	53,010	34,080	9,170	8,400	5,500	8,440	180,795
Catfish		12,735	10,867	5,273	1,115				29,990
Dogfish				760			480		1,240
Perch		200							200
Sheepshead		195							195
White bass		1,798	400						2,198
Totals	17,900	59,223	64,277	40,113	10,285	8,400	5,980	8,440	214,618

1971

Pounds								
Species	March	April	May	June	July	Aug.	Dec.	Totals
Carp	23,375	64,029	47,102	88,390	15,890		9,500	248,286
Catfish		10,183	17,745	7,951	5,120	536		41,535
Perch	22	893						915
White bass	3,066	12,058	2,129					17,253
Totals	26,463	87,163	66,976	96,341	21,010	536	9,500	307,989

TABLE 2B.6-1 (Continued)

Lake Erie Commercial Fish Production
In The Vicinity of Locust Point

1963-1972

1972

Species	March	April	May	June	July	Aug.	Dec.	Totals
				Pounds				
Bullheads				50	610	130		790
Carp	34,570	31,750	16,110	1,700	9,945	11,200	10,270	115,545
Catfish		9,079	12,660	6,040	6,845	280	470	35,374
Dogfish			2,000					2,000
Perch		67						67
White bass		2,340	4,785	1,300	110			8,535
Totals	34,570	43,236	35,555	9,090	17,510	11,610	10,740	162,311

Note: This table represents commercial production and therefore does not necessarily reflect natural populations. For example in 1970, the Ohio Division of Wildlife removed walleye from the commercial fishery and restricted the sizes of white bass which could be taken commercially because of mercury contamination.

TABLE 2B.6-1 (Continued)

Lake Erie Commercial Fish Production
In The Vicinity of Locust Point

1963-1972

Ten Year Summary

Species	1963	1964	1965	1966	1967	1968	1969	1970	1971	1972	Totals
Buffalo	20	100	37			360					517
Bullheads	120	71	24		45		35			790	1,085
Carp	74,982	10,560	49,665	179,469	78,020	151,006	103,349	180,795	248,286	115,545	1,191,623
Catfish	22,825	29,033	41,785	50,832	26,978	44,139	71,343	29,990	41,535	35,374	393,834
Dogfish								1,240		2,000	3,240
Perch	855	47	304	110	50	310	1,586	200	915	67	4,444
Quillback			12				70				82
Sheepshead	5	176	466	1,200		41	300	195			2,383
Walleye	37	199	308		135	617	1,261				2,557
White Bass	2,558	14,463	10,225	19,757	3,568	16,159	22,360	2,198	17,253	8,535	117,076
White Sucker	15	70	30			9	300				424
Totals	101,363	54,719	102,856	251,368	108,796	212,641	200,604	214,618	307,989	162,311	1,717,265
Seine Hauls	47	41	33	30	29	52	122	89	75	73	591
lbs/unit effort	2,157	1,335	3,117	8,379	3,751	4,089	1,644	2,411	4,107	2,223	2,907

TABLE 2B.6-2

Locust Point Fish Populations by Species for Months Sampled

1969-1972

Species	1969					1970						1971		1972		
	J[1]	J[2]	A[4]	S[3]	O[4]	M[4]	J[2]	J[2]	A[4]	S[2]	O[2]	A[1]	M[2]	J[1]	A[4]	S[1]
Alewife		x	x	x		x	x						x		x	
American smelt		x			x	x			x				x			
Black crappie		x	x			x				x		x	x	x	x	x
Bowfin	x	x					x									
Brown bullhead	x	x	x	x		x	x	x	x	x	x	x	x	x	x	x
Carp	x	x	x	x	x	x	x	x	x		x	x		x	x	x
Channel catfish	x	x	x	x		x	x	x	x	x	x	x		x	x	x
Coho salmon				x					x			x				
Common emerald shiner		x	x	x	x	x	x	x	x	x	x		x	x	x	
Common white sucker						x						x		x		
Freshwater drum	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
Gizzard shad	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
Golden redhorse	x			x		x					x	x	x	x		
Goldfish	x	x	x	x	x	x	x	x		x	x		x	x	x	x
Green sunfish		x														
Largemouth bass				x		x										
Logperch								x			x					
Longnose gar			x				x									
Northern pike				x		x			x							
Orange spotted sunfish		x								x	x					
Quillback	x	x	x	x		x				x	x	x	x	x	x	x
Rock bass							x									
Silver chub						x	x									
Smallmouth bass					x											
Spotted sucker						x										
Spotfin shiner		x					x									
Spottail shiner		x	x	x	x	x	x	x	x	x	x		x	x	x	
Stonecat									x							
Walleye						x		x	x			x		x	x	x
White bass	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
White crappie	x	x	x	x	x	x	x	x	x		x		x			
Yellow perch	x	x	x	x	x	x	x	x	x	x		x	x	x		
Carp X Goldfish hybrid	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
Total Species	13	20	16	18	12	23	18	14	16	13	15	14	15	16	14	11

1. commercial seine
2. commercial seine and 150-ft bag seine
3. commercial seine and gill nets
4. commercial seine, 150-ft bag seine and gill nets

TABLE 2B.6-3

Fishes Found in the Locust Point Area of Lake Erie

1963-1972

Common Name	Scientific Name	Common Name	Scientific Name
Alewife	Alosa pseudoharenqus	Largemouth bass	Micropterus s. salmoides
American smelt	Osmerus eperlanus mordax	Logperch	Percina caprodes
Black crappie	Pomoxis nigromaculatus	Longnose gar	Lepisosteus osseus
Bowfin (dog fish)	Amia calva	Northern pike	Esox lucius
Brown bullhead	Ictalurus nebulosus	Oragespotted sunfish	Lepomis humilis
Buffalo fish	Ictiobus cyprinellus	Quillback	Carpiodes cyprinus
Carp	Cyprinus carpio	Rock bass	Ambloplites rupestris
Channel catfish	Ictalurus punctatus	Silver chub	Hybopsis storeriana
Coho salmon	Oncorhynchus kisutch	Smallmouth bass	Micropterus d. dolomieui
Common emerald shiner	Notropis atherinoides	Spotted sucker	Minytrema melonops
Common white sucker	Catostomus commersoni	Spotfin shiner	Notropis spilopterus
Freshwater drum	Aplodinotus grunniens	Spottail shiner	Notropis hudsonius
Gizzard shad	Dorosoma cepedianum	Stonecat	Noturus flavus
Golden redhorse	Moxostoma erythrurum	Walleye	Stizostedion v. vitreum
Goldfish	Carassius auratus	White bass	Morone chrysops
Green sunfish	Lepomis cyanellus	White crappie	Pomoxis annularis
		Yellow perch	Perca flavescens

TABLE 2B.6-4

Relative Abundance (%) of Fish Species Sampled
in the Locust Point Area by Commercial Seine

	1969	1970						1971		1972		
Species	Oct	May	June	July	Aug	Sept	Oct	April	May	July	Aug	Sept
Alewife			<0.1						<0.1			
Brown bullhead		0.1	0.1	1.9	5.4	0.6	<0.1	<0.1	0.6	0.1	0.3	11.1
Carp[1]	48.5	95.1	95.6	64.8	11.5	92.3	82.8	98.6	85.4	14.1	8.1	28.8
Channel catfish		<0.1	0.5	1.5	18.1	2.0	0.1	<0.1		16.9	11.9	2.1
Coho salmon					7.2			<0.1				
Crappie		<0.1	<0.1	0.4	4.8	<0.1		<0.1	<0.1		<0.1	0.6
Freshwater drum		1.9	1.0	27.3	45.4	2.9		0.5	5.4	62.2	24.0	7.7
Gizzard shad	51.2	0.7	1.5	1.9		1.3	16.7	<0.1	0.8	0.6	53.1	44.2
Golden redhorse		<0.1					<0.1	<0.1	<0.1	<0.1		
Quillback		0.8				0.3	0.2	<0.1	5.8	0.9	1.0	0.6
Walleye	0.4							<0.1		2.3	0.7	2.1
White bass			0.7	1.6	2.4	0.2		<0.1	1.4	2.4	0.5	2.1
White sucker		0.3						0.1				
Yellow perch	0.1	0.1	0.1	0.1	4.8	<0.1		<0.1	0.1	<0.1		
lbs. fish/haul	113.2	2159.8	1468.4	416.2	16.5	543.3	1238.6	14188.8	1169.9	1767.5	610.4	60.3

[1] Includes Carp, Goldfish, and Carp X Goldfish hybrids

TABLE 2B.6-5

Relative Abundance (%) of Fish Species Sampled in
The Locust Point Area by Experimental Gill Nets

Species	1969			1970		1972
	Aug	Sep	Oct	May	Aug	Aug
Alewife	24.4	68.3	42.9	0.1		0.7
American smelt		<0.1	0.1			
Brown bullheads					0.8	
Carp[1]	4.6	1.5	<0.1	3.3	8.0	1.9
Channel catfish	1.1	<0.1		1.6	2.1	1.5
Crappie	0.3		0.2		0.3	
Emerald shiners	2,9	10.9		0.2	3.3	0.7
Freshwater drum	0.2	2.1	0.8	0.2	1.9	0,7
Gizzard shad	15.3	1.2	11.9	0.5	62.6	8.7
Golden redhorse	0.1					
Spottail shiners			33.6	88.8		1.1
Walleye				0.7	0.6	0.3
White bass	1.1	0.4	0.3	2.6	4.0	
Yellow Perch	49.7	15.0	9.7	1.4	16.0	82.8
No. fish/net	417.6	564.6	300.3	256.4	311.1	125.5

[1] Includes Carp, Goldfish, and Carp X Goldfish hybrids

TABLE 2B.6-6

Relative Abundance (%) of Fish Species Sampled in the Locust Point Area by 150-Ft Bag Seine

	1969			1970						1971	1972
Species	July	Aug	Oct	May	June	July	Aug	Sept	Oct	May	July
Alewife	8.0	8.0			0.5						
Black bass			<0.1								
Brown bullhead	0.2	0.2									
Carp[1]	1.6	0.8	<0.1	2.9	1.5			0.3	0.1		
Channel catfish	<0.1			0.1	0.3						
Crappie	2.7	0.3	<0.1	0.5		<0.1	0.7		0.6	0.1	0.2
Emerald shiner	13.3	20.0	0.8	7.5	69.9	1.2	8.0	18.7	0.1	95.4	73.5
Freshwater drum	<0.1			0.1		<0.1		0.7	0.1		
Gizzard shad	56.5	59.4	95.1			93.3	80.6	23.6	98.0		1.6
Smelt		<0.1					1.7			0.3	<0.1
Spottail Shiner	3.8	2.7	2.6	86.8	29.2	0.1	8.0	4.9	0.6	4.0	11.7
Sunfish	0.3							0.4	0.1		
Walleye						<0.1					2.7
White bass	12.8	8.0	1.0	1.6		3.2	0.7	15.0			9.8
Yellow perch	0.2	0.3	<0.1	0.3							
No. fish/haul	249.9	119.9	249.0	156.8	76.6	2698.8	40.9	56.6	121.0	276.9	119.3

[1] Includes Carp, Goldfish, and Carp x Goldfish hybrids

TABLE 2B.6-7

Range and Mean Total Length of Some Fish Species by Age Class Captured in the Locust Point Area of Lake Erie 1969-1971

	O	I	II	III	IV	V	VI
Alewife[2]	1.5-5.9 inches x=4.4						
Gizzard shad[2]	x=6.4	x=10.9	x=12.8	x=13.6	x=14.3		
Carp[3]	0.7-8.0 x=6.0	x=6.5	x=9.7	x=9.6	x=13.9	x=13.9	x=15.7
Goldfish[3]		x=3.5	5.0-6.0				
Quillback[3]	1.5-2.2	9.5-9.7	10.9-15.4	14.9-16.5			
Emerald shiner[1, 2]	1.7-3.1 x=2.6	2.2-3.4	3.0-3.8				
Spottail shiner[1, 2]	1.8-3.5 x=2.8	3.4-4.6	4.5-5.1	4.9-5.5			
Channel catfish[2]	x=4.8	x=7.9	x=10.1	x=12.2	x=13.2	x=15.3	x=15.9
Brown bullhead[3]	2.0-4.9	2.7-6.0					
White bass[2]	2.0-6.0 x=4.1	x=10.1	x=11.8	x=12.6	x=13.4	x=14.6	
Yellow perch[2]	2.2-4.7 x=2.6	5.1-6.9 x=6.5	6.2-7.9 x=7.2	6.5-9.8 x=8.3	7.9-10.1 x=9.0	8.6-10.6 x=9.6	
Walleye[2]	x=9.6	x=15.2	x=16.8	x=18.4	x=19.5	x=19.9	
Freshwater drum[1]	1.5-5.6 x=3.9	5.1-7.5 x=6.4	6.4-9.4 x=8.5	8.9-14.0 x=11.7	10.2-15.4 x=12.3	12.2-16.5 x=14.6	12.3-16.9 x=15.1
American smelt[2, 3]	1.6-3.6 x=2.4	5.6-6.2	x=9.5				

[1] Tubb (1972)
[2] Baker (1969)
[3] Carlander (1969)

TABLE 2B.6-8

Summary of Food Items Found in Stomachs of Fish Collected in the Locust Point Area of Lake Erie 1969-1971

Fish Species and No. Stomachs[1]		Cladocera	Copepoda	Trichoptera	Coleoptera	Diptera [2]	Chironomidae [3]	C. (Chironomus)	C. (Cryptochironomus)	Glyptotendipes	Polypedilum	Procladius	Coelotanypus	Cricotopus	Pseudochironomus	Tanytarsus	Paectrocladius	Insectra (unident.)	Amphipoda	Decapoda	Gastropoda	Pelecypoda	Fish	Unident. Debris
Gizzard shad	(144)	x	x																					x
Carp	(53)	x	x			x		x						x				x	x					x
Goldfish	(61)	x	x															x	x					x
Emerald shiner	(135)	x	x		x	x	x	x		x		x					x	x	x					x
Spottail shiner	(218)	x	x	x	x	x	x	x	x	x		x	x	x	x	x		x	x				x	x
Channel catfish	(78)	x	x	x	x	x	x	x	x	x		x	x	x				x	x				x	x
Brown bullhead	(89)	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x		x	x	x	x		x	x
White bass	(273)	x	x			x	x	x	x	x		x	x	x		x	x	x	x	x			x	x
Yellow perch	(41)	x		x	x	x	x	x	x	x								x	x			x	x	x
Freshwater drum	(98)	x	x	x		x	x	x	x	x	x	x	x	x		x		x	x	x				x

[1] Based on number of stomachs containing food.
[2] Not Chironomidae.
[3] Not identified to genus.

2B.7.0 References

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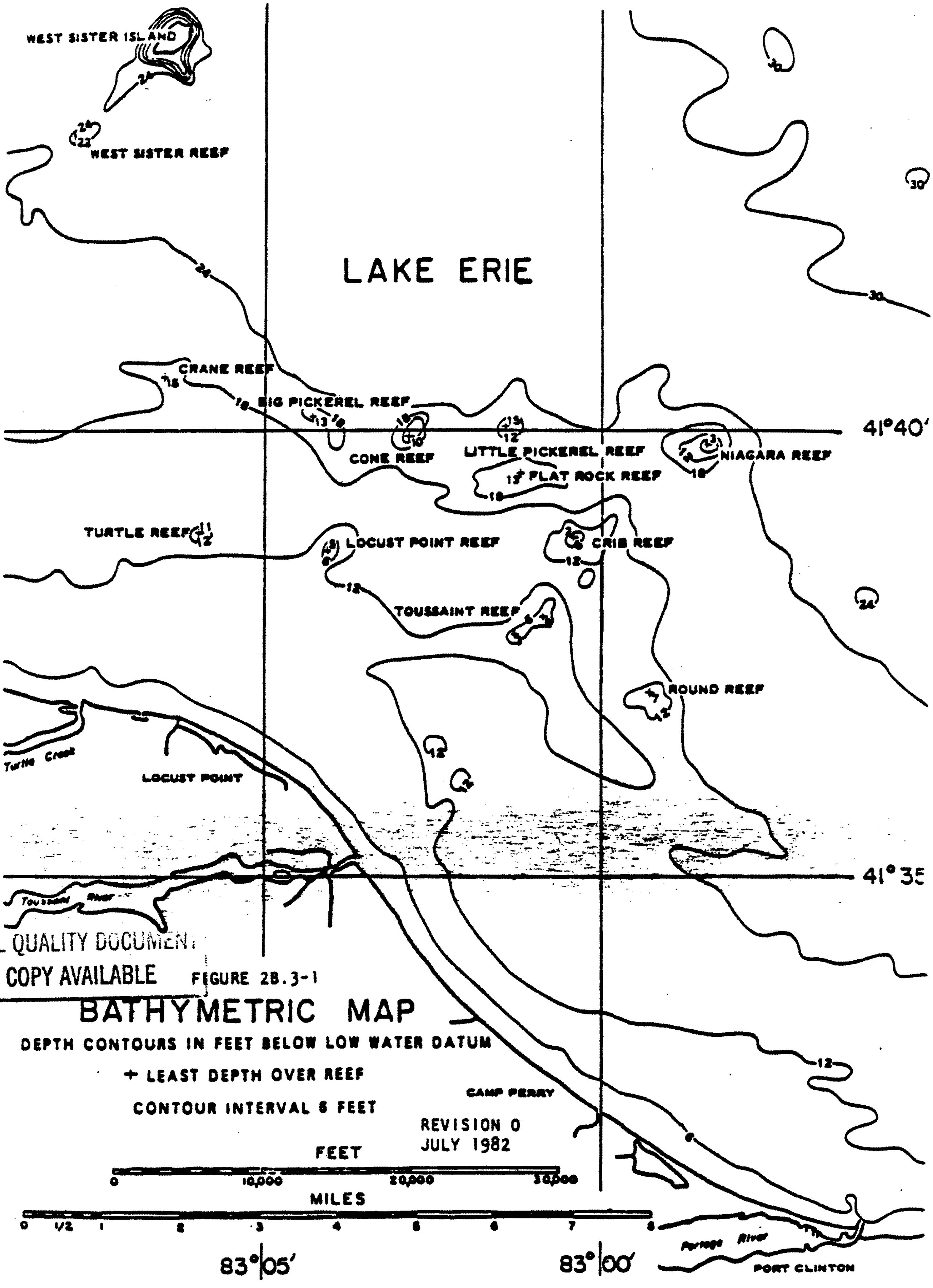

FIGURE 2B.3-1

BATHYMETRIC MAP

DEPTH CONTOURS IN FEET BELOW LOW WATER DATUM

+ LEAST DEPTH OVER REEF

CONTOUR INTERVAL 6 FEET

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JULY 1982

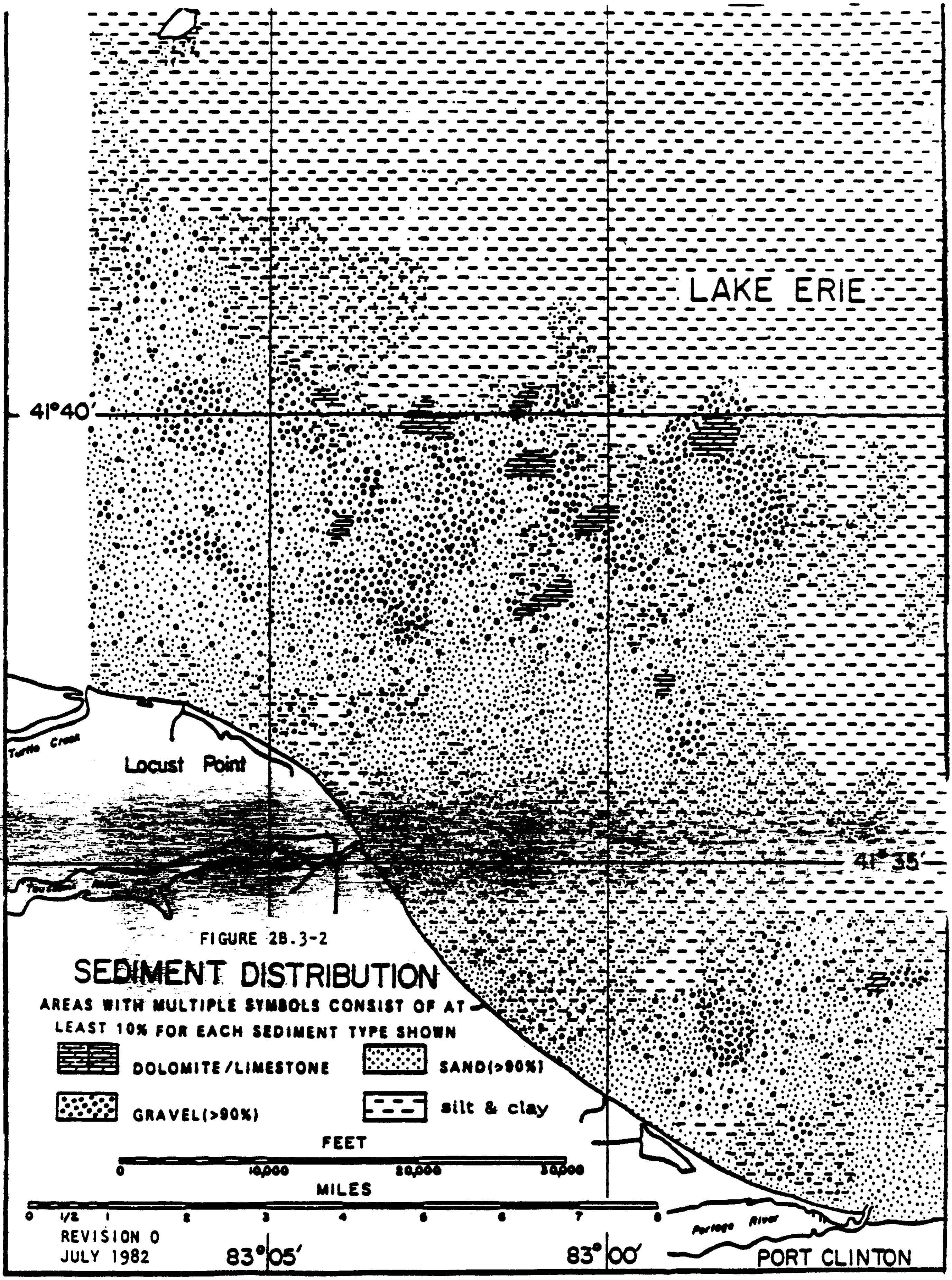
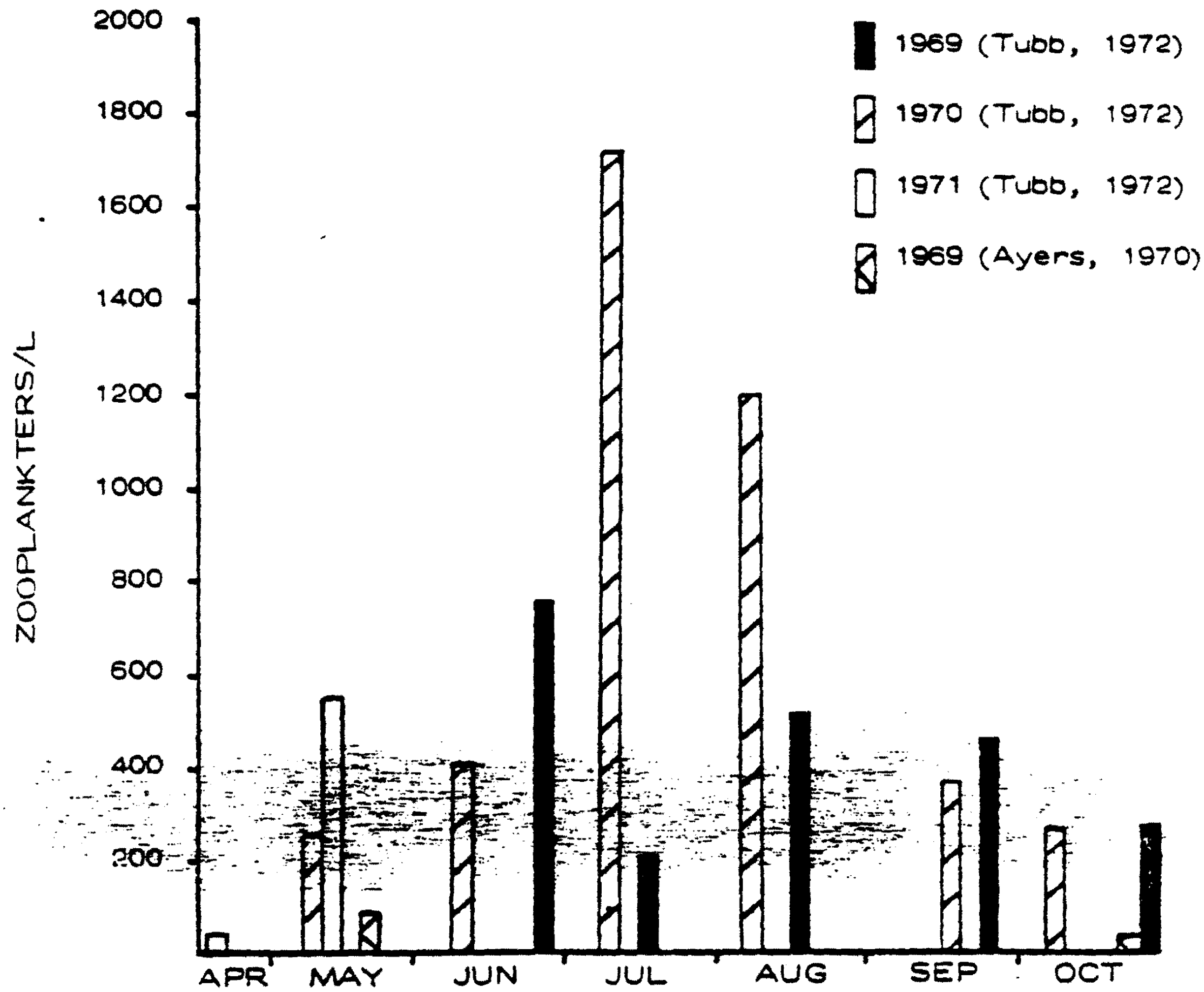


FIGURE 2B.3-2

SEDIMENT DISTRIBUTION



MEAN NUMBER OF ZOOPLANKTERS/L/MONTH - MAY, 1969 - MAY, 1971.

FIGURE 2B.4-1

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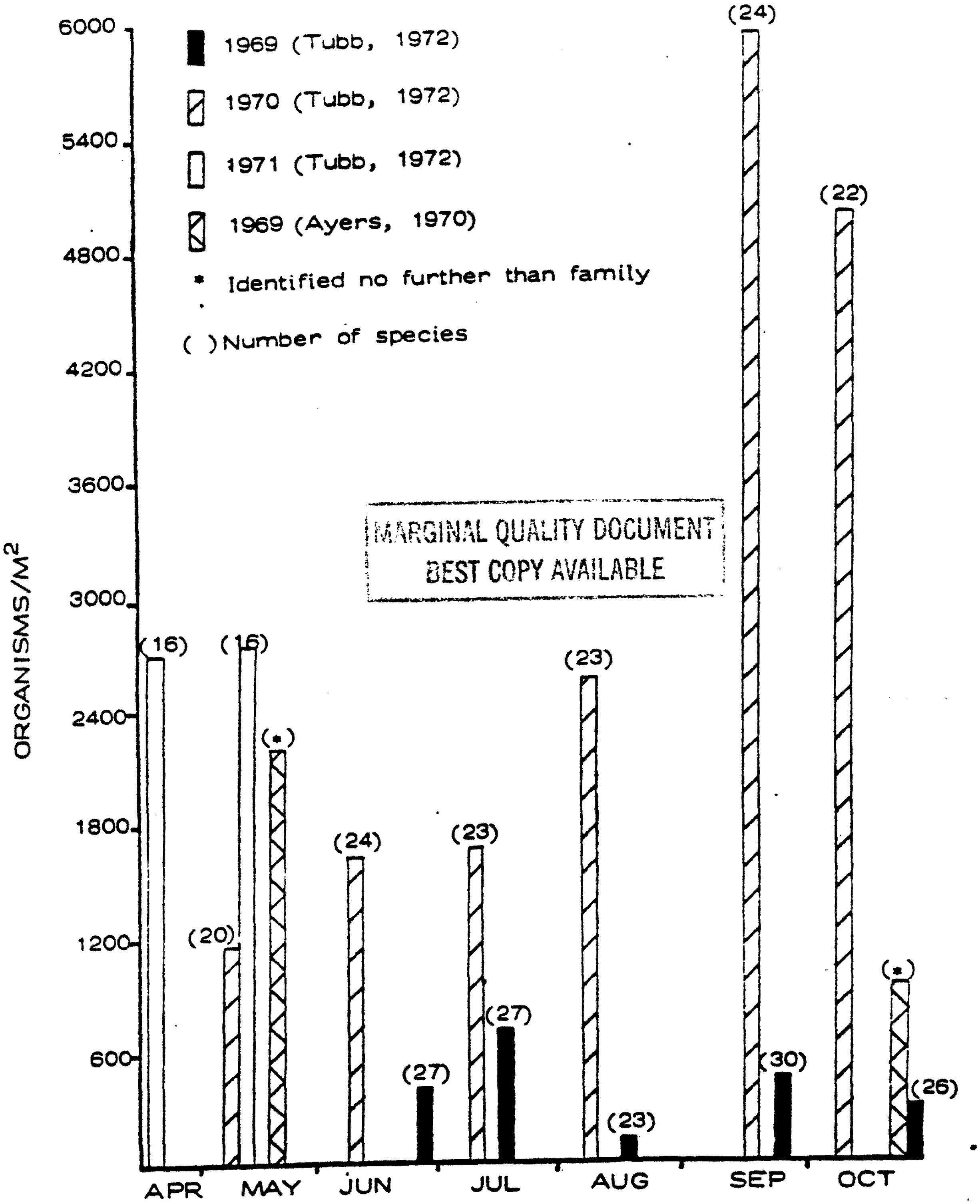
JULY 1982



MEAN NUMBER OF BENTHIC ORGANISMS/M^2/MONTH - MAY, 1969 - MAY, 1971

FIGURE 2B.4-2
REVISION 0
JULY 1982

APPENDIX 2C

GEOLOGY, SEISMOLOGY, SUBSURFACE CONDITIONS AND GEOTECHNICAL DESIGN CRITERIA

APPENDIX 2C

GEOLOGY, SEISMOLOGY, SUBSURFACE CONDITIONS AND GEOTECHNICAL DESIGN CRITERIA

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2C.1.0 Conclusion

Based on analyses of results of preconstruction and construction geotechnical investigations, it is concluded that the site on which Unit 1 of Davis-Besse Nuclear Power Station is constructed has no surface or subsurface geotechnical conditions which make it unsuitable for operation of a nuclear power station.

A study of the regional and local geology was made. This study indicated that the bedrock underlying the site consists of horizontally stratified, sedimentary, dolomitic bedrock of the Tymochtee formation. The site is on the east flank of the regional Findlay Arch. The Bowling Green fault, located 35 miles from the site at its closest point, is considered to be inactive. Faults closer to the site are not known to exist and none are suspected.

A regional and local seismologic study was made. This study indicated that earthquakes have been felt at the site with a Modified Mercalli (MM) intensity of V and that an earthquake felt at the site with the intensity of a low MM VI should be considered to have a reasonable chance of occurring during the life of the nuclear power station. Taking into account the conservatism, required for the design of a nuclear power station, it is recommended that an earthquake with the intensity of a medium MM VII (7.5) be considered to be capable of occurring at the site. Based on the seismologic study, a maximum ground acceleration of 0.08g was selected for the Maximum Probable Earthquake (the smaller earthquake), and a maximum ground acceleration of 0.15g was selected for the Maximum Possible Earthquake (the larger earthquake).

Preconstruction and construction geotechnical investigations were made. Results of the preconstruction geotechnical investigation indicated that subsurface conditions in the station area consist of 12 ft of glacial soil deposits overlying the dolomitic bedrock formation. The glacial soil consist of a stiff, fissured, desiccated, gray and brown silty clay (glaciolacustrine deposit) which overlies a hard, fissured, desiccated, gray to brown sandy clay (till deposit). The bedrock formation consists of argillaceous dolomite containing inter-bedded gypsum, anhydrite, and shale.

The bedrock formation is known to be susceptible to solution by flowing groundwater. A Bedrock Verification Program (BVP) consisting of borings, probes, geologic mapping, and geophysical surveys was implemented during foundation construction to confirm the competence of the bedrock in the station area. There was no evidence of significant solution activity detected during implementation of the BVP. Consequently, bedrock in the station area is considered to be free of significant solution activity.

Geotechnical design criteria were established for the design of foundations for station structures, for the construction of fills to raise station area grades and to form dikes, and for excavation of an Intake Canal. Foundations consist of mat or strip footings bearing on bedrock till deposit, or compacted granular fill; and pier footings socketed into bedrock. Fill construction consisted of placement and compaction of Class I and non-Class I fill in the station area, along the Intake Canal, and in the Cooling Tower Area.

The factors of safety of foundations for Class I structures and Class I embankments were determined. The minimum factor of safety for foundations beneath Class I structures (with respect to a bearing capability failure) is 4. Total settlement of Class I structures at the maximum footing contact stresses is expected to be less than 1/8 in. for structures founded in bedrock and less than 1/4 in. for structures founded in till deposit or compacted granular fill. Settlement of structures will be elastic within the range of footing contact stresses anticipated; consequently, settlements will occur upon application of the footing loads and no long-term

settlement of structures is expected. The minimum factor of safety for Class I embankments in the Intake Forebay Area (with respect to slope stability) is 1.30.

2C.2.0 Geology

2C.2.1 Introduction

A geologic study was made to analyze those aspects of the regional and local geology which are pertinent to the engineering evaluation of the site. The study consisted of a review of geologic literature, an analysis and interpretation of topographic maps and aerial photographs, interviews with local geologists, and a field geologic survey.

2C.2.2 Regional and Local Physiography

The site region is located in the Lake Plains subprovince of the Central Low Land Physiographic province. The Lake Plains subprovince is nearly flat and has poor surface drainage characteristics. The major streams in the region are the Maumee River, the Toussaint River (or Creek), and the Portage River; they generally flow toward the northeast and all drain into Lake Erie (see Figure 2C.2- 1). These streams generally have a gradient of approximately 2 ft/mi which results in low flow velocities. The surficial soils are glacial deposits. Local sedimentary bedrock exposures have a very slight dip.

The site is situated on low flat land bordering Lake Erie (see Figure 2C.2-2). The eastern portion (approximately 1/2) is marshland; the western portion (approximately 1/2) is slightly above the marsh. A narrow ridge of sand and man-placed rip rap parallels the shoreline. Lake Erie mean water level is approx El. 570.

Elevations in the text of this Appendix are with respect to the 1955 International Great Lakes Datum (IGLD). Elevations shown on Tables and Figures in this Appendix are with respect to either IGL Datum or the 1929 USC&GS Datum, and are noted accordingly. With respect to the Port Clinton benchmark, the IGL Datum[1] is 1.925 ft higher than the USC&GS Datum; therefore, the IGLD elevation of a horizontal plane is obtained by subtracting 1.925 ft from the USC&GS elevation of that plant with respect to the 1985 IGLD, the 1985 IGLD is .57 feet higher than the 1955 IGLD.

2C.2.3 Geologic History

1. Preglacial History:

Table 2C.2-1 shows the geologic time scale as used in this Appendix. During the lower and middle Cambrian epochs, the Great Lakes region was above sea level; hence, there are no deposits of these epochs. During the upper Cambrian epoch, the seas flooded the region and deposited sand. Transgression of the sea also occurred during the Ordovician period with more widespread flooding of the continent. Calcareous and clay sediments were deposited in the Great Lakes region during this period (Hough 1958).

[1] IGL Datum is hereafter referred to as IGLD in this Appendix.

In the Silurian period, a basin-and-arch pattern developed. The Appalachian Geosyncline, which had formed in early Paleozoic, continued its downsinking trend, and the Michigan Basin and Illinois Basin began to subside. More than 10,000 ft of Paleozoic sediments eventually accumulated in each basin. At the same time, other areas remained stationary or rose at intervals. These areas include the Canadian Shield and the Cincinnati Arch. The latter bifurcates along the Indiana-Ohio border, in the Anna area, into the Kankakee Arch to the northwest and the Findlay Arch to the northeast (see Figure 2C.2-3).

In the upper Silurian epoch, the seas were more restricted in area and the Michigan Basin was partially isolated from the open sea as was the Ohio-New York basin, which is the western portion of the Appalachian Geosyncline. These two basins were connected across the Findlay Arch through the Chatham Sag (also called the Ontario sag). The principal northeast in the vicinity of Georgian Bay, Ontario, connecting the Michigan Basin with the Arctic Sea. During this partial isolation from the open sea, extensive evaporitic deposits of salt and gypsum accumulated in the basins. Toward the end of the Silurian period, the seas again transgressed and usual marine calcareous sediments were deposited and subsequently lithified to dolomite.

From the Devonian through the Permian periods, there were times of alternate marine transgression and regression with a distinct trend toward less flooding each time the seas transgressed (Hough 1958). The Paleozoic era ended with Appalachian Orogeny. The interior of the continent remained essentially undisturbed during this time except for some slight warping of strata.

During the Mesozoic and Cenozoic eras, the region apparently remained above water and the predominant processes were erosion and glaciation. The age of glaciation was the Pleistocene epoch of the Quaternary period.

2. Glacial History:

Prior to glaciation, the bedrock topography in the region was one of moderate relief (Ver Steeg 1944). The valleys in western Ohio were wide and had gentle slopes such as exist in a region worn down to an advanced stage of erosion. Several glacial substages and intervening interglacial substages occurred. The glaciers invaded the region from the northeast; the sheets of ice were up to 5000 ft thick. As each glacier advanced over the region, it partially to completely obliterated evidence of preceding glaciers, picking up soil as it advanced. When the glaciers retreated, lakes formed between their ice fronts and their terminals moraines, and glaciolacustrine sediments were deposited in these lakes.

During the youngest glacial stage, Wisconsin, the lake occupying the general area of the present Lake Erie had many stages with different levels, which depended upon the outlets exposed by the advancing and retreating glacier. In general, the lake levels were higher than the present level of Lake Erie. However, during the last lake stage, which probably occurred between approximately 9000 and 12,000 years ago, the lake level was much lower than the present level of Lake Erie; that lake stage corresponds to Early Lake Erie. In the area presently occupied by the central and eastern portion of Lake Erie which is deeper than the western portion, the level of early Lake Erie is believed to have been approximately 100 ft below the present level of Lake Erie. There are indications that the area presently occupied by the western portion of Lake Erie was covered with marshes which had a water level which was approximately 30 ft to 50 ft below the present level of Lake Erie.

After the last glacier retreated, the earth crust slowly began to rise toward isostatic equilibrium (Hough 1958). This movement ceased and eventually reversed, such that subsidence is presently occurring in the entire Great Lakes region. The measured present rate of subsidence of the southern shore of Lake Erie varies between 0.5 ft/100 yr and 1.1 ft/100 yr (Moore 1948), causing small encroachment of the water on the south shore of the lake (Forsyth 1968).

2C.2.4 Regional Geology

1. Stratigraphy:

As a result of alternating times of deposition and erosion and glacial stages, the geologic strata in the region consist of glaciolacustrine deposits overlying glacial till deposits, sedimentary rocks of the Paleozoic era, and deep basement igneous and metamorphic rocks of the Precambrian era.

a. Soil strata:

The surficial soil stratum in the region is predominantly a glaciolacustrine deposit, having formed when the ice sheets retreated and outlets for lakes were considerably higher than at present. This lake deposit is generally underlain by glacial till, which varies in thickness from a few feet to over 400 ft (Goldthwait et al., 1961).

b. Sedimentary rocks:

The surficial sedimentary bedrock strata of the region consist of rocks of the Silurian through Mississippian periods. A regional bedrock map is shown in Figure 2C.2-4. Because both the Devonian and the Mississippian rocks are stratigraphically higher than the bedrock at the site, they are not further discussed.

Table 2C.2-2 is a generalized regional geologic section, with all periods after the Silurian omitted. A regional cross section, prepared from well drillers' logs, is shown in Figure 2C.2-5.

c. Igneous and metamorphic rocks:

Underlying the Cambrian strata is the basement complex of the Precambrian era. These rocks are dense crystalline granites, metamorphosed granites, metasediments, and lava flows. Figure 2C.2-3 shows the configuration of the Precambrian bedrock surface. The uppermost portion of this surface lies at approximately El. -2000 ft and dips to the north toward the Michigan Basin and to the east toward the Appalachian Geosyncline. The closest surface exposure is approximately 275 miles to the northeast. There are no known intrusive bodies of igneous rock above the Precambrian rock surface in the site region.

2. Structural Features

The major structural features in the region are the Findlay Arch, the Michigan Basin, the Appalachian Geosyncline, the Ohio-Indiana Platform, and three faults: the Bowling Green Fault, the Electric Fault, and the Osborn Fault. These features are shown in Figure 2C.2-3.

a. Findlay Arch:

The Findlay Arch is the northeastward branch of the Cincinnati Arch. The axis of the Findlay Arch is indistinct because it has a very slight warp, but it is believed to be approximately 15

miles west of the site. The crest of the arch is 10-15 miles wide. The Precambrian basement rock surface has a northwestward slope toward the Michigan Basin of approximately 1.5% and a southeastward slope toward the Appalachian Plateau of approximately 2.5%. The sedimentary strata are draped over the arch as shown in Figure 2C.2-5, and have slopes that are flatter than those of the basement rock surface.

The arch is a relative uplift that was probably created by vertical bulging caused by compressive stresses in the deeper rocks of the earth crust (Wilson 1939). This hypothesis seems to be confirmed by the core samples of the Precambrian basement rock which are gneissoid in character and show a definite alignment of minerals, indicating that the rock has been subject to intense compression (Lockett 1947). Because there is no such alignment of minerals in the Paleozoic strata, and because of the draping effect of the sedimentary rocks over the arch, it is concluded that the arch was in existence prior to the beginning of the Paleozoic era. Thus, it is concluded that the arch existed and remained unchanged during and since the Paleozoic era while the basins on either side subsided under the weight of the sediments that accumulated in them (Lockett op cit).

b. Michigan Basin:

The Michigan Basin, a saucer-shaped depression, existed throughout most of the Paleozoic era. The basin is known for the evaporitic deposits which formed during the upper Silurian epoch in the restricted sea which was periodically replenished with ocean water. The total thickness of sediments in the Michigan Basin is approximately 13,000 ft to 14,000 ft at its center and diminishes radially to approximately 3000 ft at its rim.

c. Appalachian Geosyncline:

During the Paleozoic era, the great Appalachian Geosyncline formed to the east of the site region, with thousands of feet of sediments being deposited in it. The geosyncline was generally oriented in a northeast direction. At the end of the Paleozoic era, the Appalachian Orogeny occurred, creating the present-day Appalachian Mountains.

d. Ohio-Indiana Platform:

The Ohio-Indiana Platform is a region of essentially horizontal rock strata of the lower Paleozoic era where the Cincinnati Arch bifurcates into the Kankakee and Findlay arches. The basement Precambrian bedrock is at approximately El. -2000 ft with regional dips of 0.5% to 0.7%. The Paleozoic sedimentary rocks are buried under soil, at a depth of a few feet to over 500 ft. Regional dip of the Paleozoic rocks in northeast Indiana is 0.5% toward the Michigan basin; 175 miles to the southwest, it is 0.5% toward the Illinois basin; and, between these areas there is no dip as great as 0.5% (Green 1961).

e. Bowling Green Fault:

The Bowling Green Fault, also known as the Wood County Fault, is approximately 35 miles long; it is a high-angle reverse fault striking generally north and intersecting the Findlay Arch; see Figure 2C.2-3. It has a vertical displacement of several hundred feet, with the western side downthrown. The fault is exposed in bedrock at the ground surface in Waterville, Ohio, along the Maumee River. North of Waterville, it becomes a monoclinal fold with a maximum dip of 15°; its southern extremity, as determined by drilling records, is at Findlay, Ohio. At its closest point, it is approximately 35 miles west of the site. The fault has had no apparent action in modifying the mineralogical composition of the strata (Stout 1941). The fault originated after the

upper Silurian epoch, because the youngest beds displaced are those of the Tymochtee formation (approximately 410 million years old). Both sides of the fault were leveled by erosion during the Mesozoic or Cenozoic eras. Whether this fault is genetically related to the Findlay Arch is unknown. There are no known Pleistocene deposits disturbed by the fault. It appears that no movement has occurred along the fault since the end of the Paleozoic era; therefore, the fault is considered inactive.

f. Electric Fault and Osborn Fault:

Two faults are mapped on the Tectonic Map of the United States (USGS and AAPG, 1962) in close proximity to the Chatham Sag; see Figure 2C.2-3. The Electric Fault is a normal fault with the downthrown side to the north. It has a vertical displacement of 200 ft to 300 ft and is of pre-Devonian age, probably middle to upper Silurian. The Osborn Fault is a normal fault with the downthrown side to the west. The vertical displacement is approximately 100 ft; it is known to extend to the basement rock. Its age is probably middle to upper Devonian and is certainly post-Silurian. Both these faults are considered to be inactive; at their closest points they are about 80 miles from the site.

2C.2.5 Local Geology

1. Soil Strata:

The site area, which is defined as the area within the property boundaries, is underlain by two distinct types of glacial deposits: a glaciolacustrine deposit, essentially consisting of silty clay, overlying a glacial till deposit essentially consisting of silty sandy clay. The thickness of glacial deposits in Ottawa County averages 25 ft. Within the site area, the thickness of glacial deposits was approximately 12 ft to 22 ft in the borings. A local 6- to 10-ft-thick sand ridge occurs along the lake shore and a 2- to 5-ft thick organic deposit occurs in the marsh.

In the borings made at the site, the thickness of the glaciolacustrine deposit was found to be approximately 6 ft to 12 ft. The glaciolacustrine deposit generally was found to be in a stiff condition because of desiccation, which is believed to have occurred when the level of early Lake Erie was lower than the present Lake Erie level. In the borings made at the site, the thickness of the till deposit was found to be approximately 6 ft to 12 ft. The till deposit was found to be in a hard condition. The physical properties of the soil deposits are presented in Section 2C.4 of this Appendix.

2. Rock Strata:

A map showing inferred regional bedrock surface contours and regional geology is shown is Figure 2C.2-6. The bedrock surface elevation within the site boundaries is estimated to range from El. 560 in the western portion to El. 540 in the eastern portion. The bedrock at the site has been classified as the Tymochtee formation of the Bass Island group of the upper Silurian epoch. The Tymochtee formation is a soft to hard, thin-bedded to massive, laminated, argillaceous dolomite with occasional carbonaceous shale partings along the bedding planes. It contains varying amounts of gypsum and anhydrite. The bedrock surface elevation and geological classification of bedrock in the site area were determined during construction. The results of the determinations are discussed in Section 2C.4 of this Appendix.

The Tymochtee formation has been found to be susceptible to solution activity. This characteristic is demonstrated along the western shore of South Bass Island, where the contact between the Tymochtee dolomite and the overlying Put-in-Bay dolomite is exposed a few feet

above water level. Solution activity by lake water has caused the Tymochtee dolomite to dissolve and the overlying, more resistant Put-in-Bay dolomite to collapse. Numerous caves also exist in the Tymochtee formation along the shore of the island.

At the site, some small solution fissures were observed in the preconstruction borings and rock probes and solution cavities were discovered and explored in a surface depression area 400 ft south of the station area[1]. For this reason, a Bedrock Verification Program (BVP) was implemented during construction to detect and treat any significant solution activity in the station area. The results of the BVP indicated that bedrock beneath the foundations of the station could be considered free of significant solution activity. A more complete description of the BVP is presented in Section 2C.5 of this Appendix.

Beneath the Tymochtee formation is the Greenfield formation. The lithology of the Greenfield formation is very similar to that of the Tymochtee formation, except for many carbonaceous streaks occurring as stylolites. Because of this similarity, the contact between the Tymochtee and Greenfield formations is difficult to detect. Based on results of the borings, it is concluded that the elevation of the contact is approximately El. 460 at the site.

3. Structural Features:

No faults were found in the bedrock beneath the foundations of the station. The field and literature studies did not reveal any faults in the site locality or in Ottawa County and, based on our investigation, it is concluded that none exist.

In the site locality, two well-developed joint sets are present, one treading northwest and the other northeast; the dips of both are nearly vertical (Ver Steeg 1944). Air-photo interpretation revealed the more prominent joints by inference from the subsurface “phantom” drainage pattern; their frequency and distribution are difficult to determine because they are buried under glacial deposits. During construction, exposed bedrock surfaces (both horizontal and vertical surfaces) were washed and geologically mapped to determine the characteristics of the prominent joints and to determine the nature of joint filling materials. Results of this mapping are discussed in Section 2C.4 of this Appendix.

2C.2.6 Selected References

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b. Forsyth, J. (1959) “The Beach Ridges of Northern Ohio” State of Ohio, Div of Geol Survey, Inf Circ 25

c. Goldthwait, R., White, G., and Forsyth, J. (1961) “Glacial Map of Ohio” Ohio Dept Nat Res, Div of Geol Survey

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[1] Station area in this Appendix refers to the approximate 570 ft by 750 ft area indicated in Figure 2C.4-1 and 2C.4-2.

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TABLE 2C.2-1

Geologic Time Scale

Era	Period [1]	Epoch [1]	Began Million of Years Ago [2]
Cenzoic	Quaternary	Recent	0.012
		Pleistocene	2-3
	Tertiary		65
Mesozoic			225
Paleozoic	Permian		280
	Pennsylvanian		320
	Mississippian		345
	Devonian		395
		Upper Silurian	
	Silurian		430-440
	Ordovician		500
		Upper Cambrian	
	Cambrian	Middle Cambrian	570
		Lower Cambrian	
Precambrian			3000

Notes: [1] Only periods and epochs discussed in Section 2C.2.3.1 of this Appendix are listed.

[2] Time divisions shown are those in use by the U.S. Geological Survey (1972).

TABLE 2C.2-2

Generalized Regional Geologic Section

System	Series or Group	Formation Division or Stage	Member	Kind of Material	Average Thickness in feet	Variation in Feet
Silurian	Bass Island Group	Raisin River		Dolomite, thin bedded, drab, few fossils		0-80
		Put-in-Bay		Dolomite, thin to massive bedded, gray to brown	570	200-250
		Tymochtee		Dolomite, thin to massive bedded, gray to brown		125-175 Site
		Greenfield		Dolomite, thin to massive bedded, gray to brown		175-255
	Niagara Group	Guelph		Dolomite, light, massive, very pure	80	50-100
		Cedarville		Dolomite, light to drab, massive, pure	70	50-100
		Springfield		Dolomite, drab to bluish gray, medium bedded	10	6-16
		Euphemia		Dolomite, drab to bluish gray, massive	6	2-11
		Alger	Massie	Shale, bluish gray, calcareous	5	5-6
			Laurel	Dolomite, limy, hard, medium bedded	6	5-9
			Osgood	Shale, bluish gray, calcareous	45	10-80
		Dayton		Dolomite, limy, gray to drab, medium bedded	8	7-13
	Clinton	Brassfield		Limestone, light to pink, irregularly bedded	67	20-80
				Shale, soft, variable, gray to pink	10	1-20
	Medina	Clinton		Sandstone, light to pink, fine grained	20	0-100
		Elkhorn		Shale, soft, red to variegated	63	10-200
Ordovician	Richmond	Whitewater	Upper Whitewater	Calcareous shales with thin roughly bedded limestones	75	60-100
			Saluda	Calcareous shales with thin bedded limestones		
			Lower Whitewater	Calcareous shales with thin bedded limestones		
		Liberty		Calcareous shales with some thin bedded limestones	35	30-40
		Waynesville	Blanchester	Calcareous shales with thin bedded limestones	95	85-140
			Clarksville	Calcareous shales with thin bedded limestones		
			Ft. Ancient	Calcareous shales with thin bedded limestones		
		Arnheim	Oreponia	Calcareous shales with thin bedded limestones	60	50-75
			Sunset	Calcareous shales with nodular limestones		
	Waysville	McMillan	Mt. Auburn	Calcareous shales with thin bedded limestones	90	80-120
			Corryville	Calcareous shales with thin bedded limestones		
			Bellevue	Calcareous shales with thin bedded limestones		
		Fairview	Fairmont	Calcareous shales with thin bedded limestones	115	100-150
			Mt. Hope	Calcareous shales with thin bedded limestones		

TABLE 2C.2-2 (Continued)

Generalized Regional Geologic Section

System	Series or Group	Formation Division or Stage	Member	Kind of Material	Average Thickness in feet		Variation in Feet
Ordovician	Eden	Latonia	McMicken Southgate Economy	Calcareous shales with nodular limestones Calcareous shales with nodular limestones Calcareous shales with nodular limestones	245		225-275
	Utica	Fulton		Calcareous variegated shales with small quantities of shaly limestones	170		50-400
	Trenton	Point Pleasant		Limestone or dolomite, dark, parts shaly Covered, reached only by the drill Limestone or dolomite, massive, dark	185		155-225
	Black River			Limestone or dolomite	425		375-475
	Glenwood			Dolomites with green shales	20		10-40
	St. Peter			Sandstone, local	?		0-50
	Lower Magnesian			Dolomites with some sandstone formations	450?		?-?
Cambrian				Dolomites with some massive sandstones	720?		?-?
Pre-Cambrian				Crystalline rocks, mainly gneisses and schists			

(After Stout et al, 1943)

2C.3.0 Seismology

2C.3.1 Seismicity Based on Seismic-Risk Map

As a beginning point, the seismicity of the site is evaluated on the basis of the ESSA Seismic-Risk Map of the United States (Algermissen 1969); see Figure 2C.3-1. This map shows that the site is located within Zone 1. The minimum distance from the site to a Zone 2 boundary is 40 miles and to a Zone 3 boundary, 75 miles.

Zone 1 is described as follows: "Minor damage; distant earthquakes may cause damage to structures with fundamental periods greater than 1.0 seconds; corresponds to intensities V and VI of the MM Scale" (Modified Mercalli Intensity Scale of 1931).

2C.3.2 Seismicity Based on Historic Earthquakes

1. Records of Historic Earthquakes Which Have or May Have Been Felt at the Site:

Epicenters of regional earthquakes which have or may have been felt at the site are plotted on a geologic and tectonic map; see Figure 2C.3-2. The criteria selected for plotting the epicenters are given in the legend of Figure 2C.3-2. The principal characteristics of these historically reported earthquakes are listed in Table 2C.3-1. The principal characteristics include: date and location, coordinates of the epicenter and its distance from the site, epicentral Modified Mercalli intensity at the site. The magnitude was determined by several methods as indicated in the table. Table 2C.3-1 contains two significant earthquakes not shown in Figure 2C.3-2. They are the 1886 Charleston earthquake, and the 1925 St. Lawrence Valley earthquake. The table includes 49 earthquakes which were reported between 1804 and October 1971. References used for the preparation of Figure 2C.3-2 and Table 2C.3-1 are listed in the figure. A list of selected references is given is Section 2C.3.5.

The great New Madrid earthquake of 1811-1812 had an epicentral intensity of MM XII and an epicenter 470 miles from the site, and the great Charleston earthquake of 1886 had an epicentral intensity of MM X and an epicenter 620 miles from the site. The strong St. Lawrence Valley earthquake of 1925 had an epicentral intensity of MM IX-X (9.5) and an epicenter 770 miles from the site.

Except for the above three earthquakes, the maximum epicentral intensity of the earthquakes listed in Table 2C.3-1 is MM VIII. The epicenters of the two known earthquakes of epicentral intensity MM VIII are at Attica, New York, 270 miles from the site and in Giles County, Virginia, 315 miles from the site.

Figure 2C.3-3 presents the epicentral locations and Table 2C.3-2 presents the principal characteristics of all reported earthquakes that occurred within 100 miles of the site during the period 1804 to October 1971. There are 61 such earthquakes: 21 with epicentral intensities equal to or greater than MM V and 40 with epicentral intensities less than MM V. There are eight known earthquakes with epicenters within approximately 50 miles from the site: two with epicentral intensities MM V and six with epicentral intensities MM II to MM IV.

Available in the files of the Toledo Edison Company are memoranda of interviews with local authorities and residents and newspaper accounts, which were used together with published data for the evaluation of the seismicity based on historic earthquakes.

2. Analysis of Significant Earthquakes:

a. New Madrid earthquake of 1811-1812, MM XII:

The earthquake consisted of several shocks occurring in the vicinity of New Madrid, Missouri. The larger shocks occurred on 16 December 1811 and 23 January and 7 February 1812; they have been assigned epicentral intensities of MM XII and magnitudes greater than 8. These earthquakes affected an area of approximately two million square miles (Fuller 1912). In Cincinnati, loose furniture was agitated, partition doors were opened, and church chimney tops were thrown off. The shocks were also felt in the Detroit region, but no accounts of damage are known. Based on these accounts, it is estimated that the intensity did not exceed MM V at the site.

A geologic profile of the New Madrid area shows Paleozoic limestones and sandstones underlying Mesozoic and Tertiary deposits of clay, sand, and gravel, and upper Quaternary deposits of clay, silt, and sand.

The New Madrid Fault is buried beneath the Quaternary, Tertiary, and Mesozoic deposits and cuts through the Paleozoic rocks. The origin of the New Madrid earthquakes is attributed to deep-seated movement along this fault (Fuller 1912, Nuttli 1972 personal communication).

The geology and seismic history of the New Madrid area are quite different from those of the site. The site locality contains no known active fault. It does not contain epicenters of earthquakes as large as those which occurred in the New Madrid area, nor does it contain thick soil deposits which would tend to amplify earthquake ground motions. On this basis, it is concluded that the seismicity of the site is much smaller than that of the New Madrid area.

b. Charleston earthquake of 1886, MM X:

The earthquake which occurred in the vicinity of Charleston, South Carolina, on 31 August 1886 had an epicentral intensity of MM X. It affected an area of approximately two million square miles. The earthquake consisted of two main shocks eight minutes apart. The intensity at the site is estimated to have been MM II (Woollard 1958).

A geologic profile of the Charleston area shows approximately 3000 ft of Cretaceous to Recent soil and soft rock (Thornbury 1965), beneath which is basement rock of Paleozoic and Precambrian age. Part of the city is constructed on fill; many structures are founded on piles (Dutton 1888).

The destructiveness of the earthquake was probably increased by the great thickness of soil sediments and the man-made fills. Because of different subsurface conditions, effects of earthquakes of equal magnitude are expected to be much smaller in the site locality than in the Charleston area.

c. St. Lawrence Valley earthquake of 1925, MM IX-X (9.5):

The St. Lawrence Valley earthquake of 1925 consisted of two main shocks, the first on 28 February and the second on 1 March. Both shocks had epicentral intensities of MM IX-X (9.5). The epicenter was in the St. Lawrence Valley approximately 770 mi from the site. The effects of the earthquake were felt over an unusually large area. The affected area was over one million square miles and it is probable that an additional million square miles of uninhabited northern wilderness and Atlantic Ocean bed were also shaken (Smith 1966). An isoseismal

map of this earthquake is shown in Figure 2C.3-4. The isoseismal lines are elliptical; the long axis of the elliptical curves roughly coincides with the direction of the St. Lawrence Valley. The site lies on the isoseismal line separating the MM II and MM III areas. The St. Lawrence Valley is a region of relatively high seismicity. Since 1934, six earthquakes of epicentral intensity MM IX to MM X have occurred there.

The St. Lawrence Valley is part of the Appalachian province. Recent alluvium, marine deposits, and glacial drift, sometimes several hundred feet thick, form the soil deposits. Due to folding and faulting, the bedrock geology is extremely variable. There are major north-northeast trending faults, some of which are reported to have surface expressions in the Pleistocene deposits.

Geologic evidence and seismic history both indicate that the seismicity of the site is smaller than that of the St. Lawrence Valley.

d. Anna earthquakes:

General: Several earthquakes, some with intensities of MM VII-VIII (7.5), have occurred in the area of Anna Ohio, and justify the classification of this area in Zone 2 of the ESSA Seismic-Risk map of the United States (Algermissen, op cit). From 1875 to the February 1972, there have been 31 known earthquakes in the Anna-Sidney-Lima vicinity. These earthquakes and their principal characteristics are listed in Table 2C.3-3. There are four earthquakes which are not reported by USC&GS or the Canadian Dominion Observatory, but are listed by Westland and Heinrich (Westland et al 1940). Bradley (Bradley et al 1965) reports an additional ten which are not reported by any of the preceding authors. The focal depth of the earthquakes has been estimated to be from 10 miles to 30 miles.

Anna earthquake of 2 March 1937, MM VI-VII (6.5):

The Anna, Ohio, earthquake of March 1937 had an epicentral intensity of MM VI-VII (6.5). The epicenter was at Anna approximately 100 miles from the site.

The earthquake affected an area of approximately 90,000 sq mi. The Richter magnitude based on the felt area is 5.0, that based on the epicentral intensity, 5.3.

In Anna, no serious injury was reported. The groundwater table rose a few feet. No damage was reported in Toledo. Homes shook and dishes rattled in Port Clinton. The intensity at the site is estimated to have been MM III (Westland et al, op cit).

Anna earthquake of 9 March 1937, MM VII-VIII (7.5):

The Anna, Ohio, earthquake of 9 March 1937 had an epicentral intensity of MM VII-VIII (7.5) according to the USC&GS (Neumann 1940) and MM VIII according to Bradley (Bradley at al, op cit). The area affected by the earthquake was approximately 150,000 sq mi. The instrumental Richter magnitude was 5.5 (Gutenberg et al. 1954); the Richter magnitude based on the felt area was also 5.5. Based on the epicentral intensity, the Richter magnitude was 6 if the USC&GS intensity is used and 6.3 if Bradley’s intensity is used. Both these values are greater than the magnitude of 5.5 obtained from the instrumental magnitude or based on the felt area. From this and a review of the literature, it is concluded that this Anna earthquake is conservatively characterized by an intensity of MM VII-VIII (7.5) which corresponds to a magnitude of 6.

In Anna, chimneys repaired after the 2 March earthquake were again thrown down. The groundwater table was again changed; artesian wells rose an additional few feet. In Toledo, hundreds of structures were shaken and thousands of people awakened. The intensity at the site is estimated to have been MM IV (Westland et al, op cit).

Geology of the Anna area:

The Anna area is located on the Ohio-Indiana Platform, where the Cincinnati Arch bifurcates into the Findlay Arch and the Kankakee Arch. The bedrock in the area is cut by deep erosion valleys. Glacial drift overlies bedrock with a thickness averaging 100 ft, but reaching 500 ft in local pre-glacial valleys (Rouse et al, 1938).

It is concluded that the Anna earthquakes have smaller magnitudes than calculated from their epicentral intensities because the ground motions (i.e. intensities) are amplified by the thick soil deposits which fill the preglacial valleys. This is confirmed for the 9 March 1937 earthquake for which the instrumental magnitude was appreciably smaller than the magnitude based on the epicentral intensity (5.5 vs 6.3).

Causes for the Anna earthquakes:

The earthquakes in the Anna area seem to be connected with a regional structural pattern. Dr. E. Walter, seismologist at John Carroll University, believes that the bifurcation of the Cincinnati Arch has caused a weakness in the rock and that regional strain is relieved in this area.

There is no faulting known, but there may possibly be some in the Precambrian basement rock. Deep-seated faulting connected with the Bowling Green Fault should be disregarded as the cause for the Anna earthquakes because the southern extremity of the fault is 50 miles northeast of Anna and, if the fault were extended to the south, its alignment would pass 30 miles east of Anna.

It has also been postulated that the surface rocks are rebounding as a result of glacial retreat. The resulting seismic activity is confined to a limited area of structural weakness in which most of the regional strain is relieved.

It is concluded that the Anna earthquakes have a regional cause, but occur in that area because of a local structural weakness in the bedrock, and that the effects of the earthquakes are increased because the ground motions are amplified by the thick soil deposits.

The site locality is different from the Anna area: it contains no zone structural weakness; it has no thick soil deposits; and it has a much lower seismicity based on the historic record. Therefore, the seismicity of the site is smaller than that of the Anna area.

e. Earthquakes of intensity MM VIII:

Giles County earthquake of 1897, MM VIII:

The Giles County, Virginia, earthquake of 31 May 1897 had an epicentral intensity of MM VIII. The epicenter was approximately 315 mi from the site. Based on accounts, it is estimated that the intensity at the site did not exceed MM III.

Attica earthquake of 1929, MM VIII:

The epicenter was approximately 270 miles from the site. There are no indications that the earthquake was felt at the site. On this basis, it is concluded that the intensity at the site was less than MM III.

f. Local earthquakes:

South-Central Michigan earthquake of 1947, MM VI:

The South-Central Michigan earthquake of 9 August 1947 had an epicentral intensity of MM VI. The epicenter was approximately 100 miles from the site.

In Toledo, chinaware rattled in closets and pictures swung on the walls. The Toledo Edison Company and the Ohio Fuel Gas Company received some calls, but the shock did not interfere with the service of either utility. On this basis, it is concluded that the intensity at the site was MM III.

Cleveland earthquake of 1928, MM V:

The Cleveland, Ohio, earthquake of 9 September 1928 had an epicentral intensity of MM V. At the time of the earthquake, airplanes were engaged in bombing practice over the lake and near Camp Perry (Heck et al 1930), which is 4.5 miles southeast of site. Whatever the cause of the shocks, it is concluded that the site was not subjected to intensities exceeding MM V.

Earthquakes within 50 miles of the site:

There are eight known earthquakes within 50 miles of the site, of which two each had epicentral intensities of MM V, MM IV, MM III, and MM II. The closest earthquake of epicentral intensity MM V had its epicenter 30 miles from the site.

3. Discussion:

a. New Madrid - St. Lawrence Valley Reported Belt of Earthquake Epicenters:

Several authors have reported an alignment, zone of earthquake epicenters, trend, etc, along the general direction defined by New Madrid and the St. Lawrence Valley. This belt has been identified on the basis of three main arguments:

1. There appears to be a general alignment of earthquake epicenters extending from New Madrid, up the Mississippi Valley, across Indiana, the Anna area, Lake Erie, Lake Ontario, and the St. Lawrence Valley. This alignment has been reported by several authors (Bradley et al 1965; Hodgson 1964; King 1965; Richter 1959; USC&GS 1931; Westland et al 1940; Wilson et al 1958; Woollard 1958). If the cluster of epicenters in the Anna area (which may be explained by a localized structural weakness in the bedrock) is not considered, the apparent alignment of epicenters between New Madrid and the St. Lawrence Valley is weakened.

2. The direction of the New Madrid - St. Lawrence Valley reported belt of earthquake epicenters coincides with a direction of better-than-normal seismic transmissibility as evidenced by the isoseismals of several earthquakes. Figure 2C.3-4 shows that the long axis of the isoseismals of the St. Lawrence Valley earthquake of 1925 is

parallel to the reported belt of earthquake epicenters. This direction of better-than-normal seismic transmissibility has been reported by several authors (King op cit; Woollard op cit).

3. A structural geologic reason has been postulated for the reported belt of earthquake epicenters. The eastern front of the Grenville orogeny appears to have the same general direction as the reported belt (Snyder 1968). The location of this front has been inferred on the basis of age dating of the Precambrian basement rock; no Precambrian fault has been identified by more reliable methods, such as gravimetry and borings. There is evidence of faulting in upper Paleozoic and Mesozoic deposits in both the New Madrid and St. Lawrence Valley areas; between these areas, there are no known faults in the Paleozoic rocks. Therefore, the argument for the belt existence, based on structural geology, is weak.

On the basis of the above arguments, the possibility that the New Madrid - St. Lawrence reported belt of earthquake epicenters could be identified with a tectonic structure or a tectonic province was examined. This reported belt is not a tectonic structure because no continuous large-scale dislocation or disruption within the earth's crust along the direction of this apparent belt has been reported or is suspected to exist. This is not the case for either the New Madrid or St. Lawrence Valley areas where local faults cutting the upper Paleozoic and Mesozoic exist and have been identified with earthquakes. This reported belt is not a tectonic province. The New Madrid area is in the Mississippi embayment of the Gulf Coastal Plain, the St. Lawrence Valley area is in a rift zone, the site is on the Findlay Arch. There is no unity of geologic structural features in this reported belt.

On the basis of historic records and structural geologic features, it is concluded that the seismicity of both the New Madrid and the St. Lawrence Valley areas is much greater than the seismicity of the several geologic areas which are located between these two areas. It is also concluded that the New Madrid - St. Lawrence reported belt of earthquake epicenters does not influence the seismicity of the site.

b. Shocks felt by local residents:

Ten local residents were interviewed and asked to report any recollection of tremors that they could attribute to earthquakes. None of the persons interviewed had any recollection of having felt earthquake tremors in the site locality. Many reported ground shaking when Camp Perry was firing heavy artillery. The shocks from Camp Perry were severe enough to crack plaster and water cisterns.

Camp Perry was opened in 1918, approximately 4.5 miles southeast of the site. The maximum shell size fired prior to 1965 was eight inches. A maximum shell size of 40 mm was occasionally fired until anti-aircraft training firing was suspended in 1988.

It is concluded that, if shocks felt by the local residents were earthquake tremors, their intensity did not exceed that of the shocks caused by heavy artillery to which they were accustomed. This is a confirmation that the earthquake intensity felt at the site has not exceeded MM V during the last half century.

4. Conclusion:

Historic records indicate that earthquakes have never been felt at the site with an intensity greater than MM V, and that no earthquakes of epicentral intensity greater than MM V have

occurred within 50 miles of the site. There is no reason to believe that the seismicity of the site will change.

As a group, the intensity of the New Madrid (1811-1812), the Charleston (1886), and the St. Lawrence Valley (1925) earthquakes was MM IX-X (9.5) to MM XII. Study of these earthquakes shows that the geology, including soil conditions, in their epicentral areas differs from the site geology and that the site seismicity is much smaller than the seismicity in the epicentral areas of these earthquakes.

Study of the Anna earthquakes, which had maximum intensity of MM VII-VIII (7.5), indicates that they can be attributed to a local structural weakness in the bedrock and that their effects probably are amplified by the thick soil deposits. Because these conditions do not exist at the site, the seismicity of the site is smaller than that of Anna.

Based on the study of the historic regional and local earthquakes, it is concluded that earthquakes felt at the site with the intensity of a low MM VI should be considered to have a small probability of occurring, and that it is improbable, but possible, that earthquakes be felt at the site with the intensity of a medium MM VII (7.5).

2C.3.3 Influence of Regional and Local Geologic Structural Features on Seismicity

1. Discussion:

The regional and local geologic structural features which have or may have some effect on the seismicity are discussed below.

The Ohio-Indiana Platform is south of the site, where the Cincinnati Arch bifurcates into the Kankakee Arch and the Findlay Arch, and where the Anna earthquakes occurred.

The only significant regional fault is the Bowling Green Fault which is approximately 35 miles from the site at its closest point. It is concluded that the Anna earthquakes are not related to displacements along the Bowling Green Fault, because the southern extremity of this fault is reported to be near Findlay, Ohio, approximately 50 miles from Anna, and the Anna area is not in line with the general direction of this fault. No evidence of displacement along this fault younger than the Silurian period has been found during the geologic study. On this basis, it is concluded that the Bowling Green Fault is inactive.

The site region is adjacent to the stable Canadian Shield. Some seismologists, e.g. Richter (Richter 1959), indicate that stable shields are often fringed by belts of moderate seismicity with occasional strong earthquakes. There is no indication from historic earthquakes that the site region is in one of these belts.

The site region is part of the Great Lakes region, in which earthquakes may be caused by bedrock rebound subsequent to retreat of the glaciers. Only two relatively small earthquakes (epicentral intensity MM VI) may be explained by this cause.

The site is underlain by the Findlay Arch which formed and ceased its formation prior to the Paleozoic era. However, considering the earthquake epicenter map of the region, there appears to be a concentration of epicenters of small to moderate earthquakes on the Findlay Arch (see Figure 2C.3-3). There are no topographic expressions at the site which could possibly be related to earthquakes. The subsurface investigation did not disclose any fractures which could be interpreted as resulting from tectonic movements.

Earthquake epicenters in the Anna area are located in approximately the same area delineated by the bifurcation of the Cincinnati Arch. The confluence of these three different structural trends (Cincinnati Arch, Kankakee Arch, and Findlay Arch) would indicate the occurrence of some structural disturbance, anomaly, or weakness.

Drs. Walter (Walter, 1973), Sbar (Sbar, 1973), Nuttli (Nuttli, 1973), Mancusco (Mancusco, 1973), Jansen (Jansen, 1973), Clifford (Clifford, 1973), Andersen (Andersen, 1973), Frank (Frank, 1973), Pawlowicz (Pawlowicz, 1973) and Fr. Bradley (Fr. Bradley, 1973) concur that the hypothesis of a local structural disturbance, anomaly, or weakness in the Anna area having a causal relationship with the earthquake epicenter in the Anna area is logical.

In addition, Mayhew (Mayhew, 1969) has postulated the existence of subsurface fault traces in the Anna area based on seismic reflection studies, and Pawlowicz (Pawlowicz, 1973) indicates a preliminary correlation between the nearby Lima, Ohio oil field withdrawals and the Anna earthquakes. Such a correlation indicates the existence of local subsurface weakness or faults which are responding to stress changes initiated by oil and gas withdrawal.

There is no seismic or geologic evidence to indicate any relationship between the postulated local structural weaknesses in the Anna area with the Findlay Arch in general or the Davis-Besse site area in particular. The Findlay Arch is a Precambrian basement rock structure with a very broad crest width of 10 to 15 miles, and very gentle side slopes (typical slopes of 1.5 percent to 2.5 percent); subsequent sedimentary strata draped over the arch have bedding slopes that are even flatter than those of the arch. There is no evidence of faulting, severe warping, or fracturing of the Arch or overlying sedimentary rocks. The Davis-Besse site is located on the east slope of the Findlay Arch about 75 miles northeast of Anna area and the area of the bifurcation. Because there is no significant historical seismic activity correlated to the arch except at the bifurcation point and there is no direct or indirect evidence for structural weakness, faults, or fracturing except in the bifurcation area, we conclude that there is no basis to conclude strain release might occur at the Davis-Besse site.

2. Conclusions:

The regional geologic study has disclosed regional geologic features which affect or may affect the seismicity of the region or localized areas in the region. The Findlay Arch, which may be associated with small to moderate earthquakes, underlies the site. The Ohio-Indiana Platform affects the Anna area, but not the site locality. The Bowling Green Fault, approximately 35 miles from the site, is considered inactive. The site region and locality lie in the fringe of the Canadian Shield and, while there is no evidence that they are in a seismically active belt, they may be subjected to rebound due to glacial retreat.

The local geologic study, the examination of the local topography, and the site subsurface investigation have not disclosed any local geologic features which would tend to affect the seismicity of the site locality. No local faults have been recognized and none are thought to exist.

The seismicity of the site may be affected by the Findlay arch. It is not affected by other regional geologic structural features, and it is not affected by local geologic structural features.

2C.3.4 Determination of Design Earthquakes

1. General:

Two design earthquakes are recommended: the Maximum Possible Earthquake and the Maximum Probable Earthquake. The Maximum Possible Earthquake (larger) produces a vibratory ground motion for which structures, systems, and components important to safety are designed to remain functional. The Maximum Possible Earthquake and associated ground motion induces the maximum vibratory ground motions into rock-like material at the site which, under the presently known existing geologic conditions, could conceivably or possibly occur at the site. The Maximum Possible Earthquake is similar to the Safe Shutdown Earthquake terminology presently being used. The Maximum Probable Earthquake (smaller) produces the vibratory ground motions used in the design of structures and equipment whose failure would not result in the release of significant radioactivity and would not prevent reactor shutdown. The Maximum Probable Earthquake and associated ground motion induces the maximum vibratory ground motions into rock-like material at the site which, under the presently known existing geologic conditions, might, with small chance, reasonably or probably be expected to occur at the site. The Maximum Probable Earthquake is similar to the Operating Basis Earthquake terminology presently being used. The Maximum Possible Earthquake is selected primarily on the basis of structural geologic features. The Maximum Probable Earthquake is selected primarily on the basis of the historic earthquakes with consideration, at least in a qualitative way, of the probability of occurring. This USAR terminology to define the earthquakes has been adopted to provide continuity between the various rationale, criteria, and design commitments made in the PSAR and the final design evaluations presented in the USAR.

These two design earthquakes are selected independently on the basis of seismology only. Determination of the design earthquakes is based on the ESSA Seismic-Risk Map, the records of the historic earthquakes, and the regional and local geologic structural features. Determination of a design earthquake for structural analysis includes the determination of the following parameters: maximum vibratory ground acceleration, maximum vibratory ground velocity, maximum vibratory ground displacement, and total duration of the vibratory ground motions. An accelerogram of the vibratory ground motions (ie, a plot of acceleration versus time) is also selected.

Several authors have developed equations (eg, Gutenberg et al 1942; Hershberger 1956; Esteva et al 1964) and charts (Seed et al 1969) which may be used as a guide for determination of design earthquake parameters. In addition, Professor N. M. Newmark (Newmark et al 1969, Table I) has suggested standard relative values of maximum ground acceleration, velocity, and displacement.

2. Determination of the Characteristics of the Maximum Probable Earthquake (smaller earthquake):

a. Determination of maximum horizontal vibratory ground acceleration, velocity, and displacement:

The study of the site seismicity suggests the selection of several earthquakes for the determination of the parameters of the Maximum Probable (smaller) Earthquake: a great, distant earthquake (earthquake A), a moderately strong earthquake in the Anna area (earthquake B), and a local earthquake (earthquake C).

Earthquake A: Earthquake A is conservatively modeled after the New Madrid, Charleston, and St. Lawrence Valley earthquakes. The parameters of these earthquakes and those of earthquake A are given below.

Earthquake	Epicentral Intensity MM	Magnitude	Distance From Epicenter To Site, Miles
New Madrid	XII	8	470
Charleston	X	7.1 - 7.7	620
St Lawrence Valley	IX-X (9.5)	6.4 - 7.3	770
Earthquake A	XII	8	400

Earthquake B: Earthquake B is conservatively modeled after the Anna earthquakes. The parameters of the stronger Anna earthquake (that of 9 March 1937) and those of earthquake B are given below.

Earthquake	Epicentral Intensity MM	Magnitude	Distance From Epicenter To Site, Miles
Anna	VII-VIII (7.5)	6	100
Earthquake B	VII-VIII (7.5)	6	75

Earthquake C: Earthquake C is conservatively modeled after the Lake Erie and South-Central Michigan earthquakes. For the purpose of selecting the parameters of the Maximum Probable (smaller) Earthquake, it is considered that during the life of the station, the epicenter of such an earthquake has a reasonably small chance of being located at a minimum distance of, say, 30 mi from the site. The parameters of the Lake Erie and South-Central Michigan earthquakes and those of earthquake C are given below.

Earthquake	Epicentral Intensity MM	Magnitude	Distance From Epicenter To Site, Miles
Lake Erie	V	3.7- 5.5	115
South-Central Michigan	VI	4.5 - 5	100
Earthquake C	VI	5.3	30

The maximum ground accelerations, velocities, and displacements induced at the site by earthquakes A, B, and C are calculated from Esteva and Rosenblueth's equations assuming a focal depth of a few miles.

Earthquake	Acceleration A/g	Velocity in/sec	Displacement in.
A	0.01	0.3	0.04
B	0.02	0.6	0.08
C	0.06	1.6	0.19

The maximum acceleration, velocity, and displacement at the site is induced by earthquake C. The acceleration of 0.06g is calculated from Esteva and Rosenblueth's equation for an epicentral distance of 30 miles. This distance is approximately the minimum distance for which the equation is applicable. For smaller epicentral distances, the acceleration can be calculated from the Hershberger, Gutenberg and Richter, and TID 7024 equations. These latter equations

give accelerations smaller than 0.06g for an earthquake of epicentral intensity equal to a low MM VI.

Historically, no earthquakes have been felt at the site with an intensity greater than MM V, which corresponds to a maximum ground acceleration somewhat less than 0.02g. However, the seismicity study indicates that earthquakes felt at the site with an intensity of a low MM VI should be considered to have a small probability of occurring. Such an intensity corresponds to a maximum ground acceleration of 0.06g. An acceleration of 0.06g is approximately three times the estimated historic maximum vibratory ground acceleration which has occurred at the site and is, therefore, considered conservative and reasonable. However, based on the AEC design requirements in effect when the Maximum Probable (smaller) Earthquake was selected, the maximum horizontal ground acceleration of the Maximum Probable (smaller) Earthquake was increased to 0.08g.

The maximum horizontal vibratory ground velocity and displacement were determined on the basis of the standard relative values suggested by Professor Newmark including his suggestion that these standard relative values be multiplied by 0.67 when the structures are founded on competent rock; however, the maximum horizontal ground acceleration value of 0.08g was not reduced in this manner. A maximum horizontal vibratory ground displacement of 1.78 in. was selected for the Maximum Probable (smaller) Earthquake.

b. Determination of maximum vertical vibratory ground acceleration, velocity, and displacement:

The maximum vertical vibratory ground acceleration, velocity, and displacement of the Maximum Probable (smaller) Earthquake were two-thirds of the corresponding parameters of the horizontal vibratory ground motions.

c. Determination of duration of vibratory ground motions:

The seismicity study indicates that the total duration of the vibratory ground motions occurring during earthquakes in the site region and locality appears to be typically 30 seconds and that the duration of the strong motions does not exceed a few seconds. On this basis, the Maximum Probable (smaller) Earthquake has a total duration of approximately 30 seconds with the duration of its strong motions being approximately 3 seconds.

3. Determination of the Characteristics of the Maximum Possible Earthquake (larger earthquake):

a. Determination of maximum horizontal vibratory ground acceleration, velocity, and displacement:

The study of the site seismicity suggests the selection of two earthquakes for the determination of the parameters of the Maximum Possible (larger) Earthquake: a strong earthquake in the Anna area (earthquake D) and an earthquake close to the site (earthquake E).

Earthquake D: Historic earthquakes in the Anna area had epicentral intensities MM VII-VIII (7.5). For the purpose of selecting the parameters of the Maximum Possible (larger) Earthquake, it was considered that in the Anna area, an earthquake stronger than the historic earthquakes is capable of occurring. Earthquake D is modeled conservatively after an earthquake of epicentral intensity MM VIII-IX (8.5), magnitude 6.7, the epicenter of which is 75 miles from the site.

According the Esteva and Rosenblueth's a equations, earthquake D induces at the site a maximum ground acceleration of 0.04g a maximum velocity of 1.4 in/sec, and a maximum displacement of 0.18 in.

Earthquake E: For the purpose of selecting the parameters of the Maximum Possible (larger) Earthquake, the possibility of an earthquake occurring close to the site was considered. Two possible cases are considered for the selection of the parameters of earthquake E.

1. Lake Erie and South-Central Michigan earthquakes. It was conservatively assumed that the largest possible earthquake of this type could have an epicentral intensity of MM VI and could have an epicenter close to the site.

2. Anna earthquakes. Both the site and the Anna area are located on the Findlay Arch. It was conservatively assumed that an earthquake of epicentral intensity equal to a medium MM VII (7.5), similar to the Anna earthquake of 1937, could possibly have an epicenter close to the site.

The maximum acceleration induced at the site by earthquake E is estimated to be 0.15g. This value of acceleration is slightly smaller than that calculated from the Hershberger equation for an earthquake of intensity equal to a medium MM VII (7.5) and slightly larger than those calculated from the Gutenberg and Richter equation and the TID 7024 equation. Taking into account that bedrock is encountered at the site at shallow depth and that the above equations were developed for average conditions not as favorable as those encountered at the site, it was concluded that the estimated maximum ground acceleration of 0.15g is both conservative and reasonable. Therefore, a maximum horizontal vibratory ground acceleration of 0.15g was selected for the Maximum Possible (larger) Earthquake.

The maximum horizontal vibratory ground velocity and displacement were determined on the basis of the standard relative values suggested by Professor Newmark, including his suggestion that these standard relative values be multiplied by 0.67 when the structures are founded on competent rock; however, the maximum horizontal ground acceleration value of 0.15g was not reduced in this manner. A maximum vibratory horizontal ground velocity of 5.0 in/sec and a maximum horizontal vibratory ground displacement of 3.33 in. were selected for the Maximum Possible (larger) Earthquake.

b. Determination of maximum vertical vibratory ground acceleration, velocity, and displacement:

The maximum vertical vibratory ground acceleration, velocity, and displacement of the Maximum Possible (larger) Earthquake were two-thirds of the corresponding parameters of the horizontal vibratory ground motions.

c. Determination of duration of vibratory ground motions:

As in the case for the Maximum Probable (smaller) Earthquake, the Maximum Possible (larger) Earthquake has a total duration of approximately 30 seconds with the duration of its strong motions being approximately 3 seconds.

4. Determination of the Accelerograms for the Vibratory Ground Motions of the Design Earthquakes:

a. East-west accelerogram of 31 October 1935 Helena earthquake:

The accelerogram of the east-west component of the Helena, Montana, earthquake of 31 October 1935 is used as the basis for the development of the accelerograms of the Maximum Probable (smaller) Earthquake and Maximum Possible (larger) Earthquake. A record giving acceleration versus time was obtained from the original accelerogram (Cloud, personal communications). Appropriate base line corrections were applied and the record was redigitized at intervals of 0.01 seconds for a duration of 10 seconds by Dr. I. M. Idriss and provided by Professor H. Bolton Seed of the University of California.

The development of the accelerograms for horizontal vibratory ground motions for both the Maximum Probable (smaller) Earthquake and Maximum Possible (larger) Earthquake is described in Table 2C.3-4.

b. Basis for selection of the 31 October 1935 Helena earthquake:

The Helena earthquake of 31 October 1935 is selected because many of its characteristics are similar to those which would be expected of the Maximum Possible (larger) Earthquake. The predominant period of the Helena record is equal to the predominant period that would be associated with the Maximum Possible (larger) Earthquake which is estimated from graphs recently published (Seed et al 1969). These graphs indicate that the predominant period for a magnitude 6 local earthquake is about 0.25 seconds. Another accelerogram with a similar predominant period is that recorded at Golden Gate Park during the 22 March 1957 San Francisco earthquake. However, an examination of the response spectra of the Helena and Golden Gate records indicate that the spectral values of the Helena record are higher than those of the Golden Gate record for long periods (i.e., longer than 0.5 seconds; see Figure 2C.3-7). Such a characteristic is expected from earthquakes in the site region. Thus, the Helena record provides a more applicable and more conservative estimate of the anticipated ground motions at the site than would the Golden Gate record.

The characteristics of the Helena record are more applicable to the site than are the characteristics of the record obtained at El Centro during the 18 May 1940 Imperial Valley earthquake, or the record obtained at Taft during the 21 July 1952 Kern County earthquake. Both the Imperial Valley and Kern County earthquakes were stronger than the selected Maximum Possible (larger) Earthquake. Their accelerograms were recorded by soil-supported seismographs, whereas the Helena accelerogram was recorded by a rock-supported seismograph; thus, the Helena record is more applicable for estimating ground motions of competent rock.

The characteristics of the proposed ground motions are those of the Helena, Golden Gate, Taft, and El Centro records, together with the known site conditions at the recording stations, are compared in Table 2C.3-5.

c. Calculation of the response spectra of the east-west component of the 31 October 1935 Helena earthquake:

Significant discrepancies were found among published response spectra of the Helena earthquake. For example, for zero damping, one response spectrum gave the maximum acceleration (i.e., 1.5 g) and maximum velocity (i.e., 20 in/sec) for a frequency of 4.5 cy/sec;

whereas another response spectrum gave the maximum acceleration (i.e., 0.9g) for a frequency of 5.5 cy/sec and the maximum velocity (i.e., 12.5 in/sec) for a frequency of 2.5 cy/sec.

The response spectra of the selected accelerogram of the Helena earthquake were calculated for several damping ratios by means of a program recently developed at the University of California (Idriss et al 1969). These response spectra were checked and found very similar to the response spectra of the east-west component of the 31 October 1935 Helena earthquake calculated independently at the Massachusetts Institute of Technology (Whitman, personal communication). A comparison between these two independently calculated response spectra is shown in Figure 2C.3-8.

d. Design response spectra:

The design response spectra for horizontal vibratory ground motions of the Maximum Possible (larger) Earthquake for damping ratios of 0, 0.005, 0.01, 0.02, 0.05, and 0.1 are plotted in 2C.3-5.

Design response spectra for horizontal vibratory ground motions of the Maximum Probable (smaller) Earthquake are obtained by multiplying the corresponding response spectra for the Maximum Possible (larger) Earthquake by a factor of 8/15. The design response spectra for horizontal vibratory ground motions of the Maximum Probable (smaller) Earthquake are shown in Figure 2C.3-6.

Response spectra were determined for structures founded on rock-like material; i.e., the bedrock or till deposit underlying the site. The response spectra of the Maximum Possible (larger) Earthquake corresponds to the maximum vibratory ground motions at the elevations of the foundations of the station. These response spectra were developed considering such foundations to be single-degree-of-freedom damped oscillators and neglecting soil-structure interaction effects.

Response spectra were developed for the Davis-Besse Station by considering the Helena response spectrum and several often used average response spectra. Average response spectra are developed from several response spectra of individual earthquakes. Consideration of average response spectra is made to remove the peculiarities of any response spectrum calculated from a single earthquake. For example, the response spectrum of the Helena earthquake has a valley (low response) for frequencies in the vicinity of 2 cy/sec; for these frequencies, the Helens response is not used and average response spectra are considered.

The design response spectra were developed from the ground motions given in Table 2C.3-4, using Professor N. M. Newmark's suggested method (Newmark et al, op cit, p. B-4, 43-45), except for one modification.

The spectral amplification factors suggested by Professor Newmark (Newmark et al, op cit. Fig. 2) were modified for two reasons (1) the maximum ground accelerations were not reduced by the factor 0.67 as it is suggested by Professor Newmark when the foundation conditions consist of competent rock; and (2) consideration was made of the response spectrum of the Helena earthquake.

The design maximum ground accelerations were not reduced by the factor 0.67 before developing the response spectra because a comparison of the Newmark reduced response spectra and the Helena response spectra showed that the Newmark reduced response spectra were below (less conservative) the Helena response spectra for frequencies in the frequency

range of major reinforced concrete structures. Plotted in Figure 2C.3-7 are the Newmark response spectra (unreduced and reduced) and the Helena response spectrum for zero damping. The Helena response spectrum lies between the two Newmark response spectra in the frequency range 5.5 to 6 cy/sec.

The spectral amplification factors are given in column (9) of Table 2C.3-6. These amplification factors were used for the development of the design response spectra of the synthesized earthquakes shown in Figures 2C.3-5 and 2C.3-6.

Table 2C.3-6 contains amplification factors for spectral accelerations, velocities, and displacements determined by different approaches for several damping ratios. Amplification factors for spectral acceleration at a high frequency (i.e., 5 cy/sec) are listed in part (a) of the table. Amplification factors for spectral velocity at a medium frequency (i.e., 1 cy/sec) are listed in part (b) of the table. And, amplification factors for spectral displacement at a low frequency (i.e., 0.2 cy/sec) are listed in part (c) of the table.

Column (1) of the table contains the damping ratios for which the amplification factors were calculated.

Column (2) contains amplification factors calculated for an earthquake of 30-seconds duration using rules developed by Professors L. Esteva and E. Rosenblueth and reported by Professor R. Whitman (Whitman, personal communication). Column (2) is given for comparison only, because these rules were developed for distant earthquakes not quite applicable to the site.

Column (3) contains amplification factors calculated from the curves plotted in TID 7024 (USAEC 1963, p. 1.3D, Fig. 1.21). These curves were developed by Professor G. Housner on the basis of the relative velocity spectra of several earthquakes.

Column (4) contains amplification factors recently calculated by Professor H. Amin at the University of Illinois (Amin 1968; Amin, personal communication). These factors were calculated considering the average value of pseudo-velocity spectra of the same earthquakes considered by Professor Housner. Professor Housner's relative velocity spectra were calculated by means of an analog computer whereas Professor Amin's pseudo-velocity spectra were recently calculated by means of a digital computer. The design response spectra, as well as Professor Amin's, are calculated in terms of pseudo-velocity; whereas Professor Housner's response spectra are calculated in terms of relative velocity. For certain frequencies, this introduces significant differences, as has been shown by Professors A. S. Veletsos and Newmark (Veletsos et al 1964). The amplification factors for spectral acceleration calculated by Professor Amin are greater (more conservative) than those calculated from Professor Housner's curves.

Columns (5) and (6) contain amplification factors that were obtained by smoothing (local averaging) the response spectrum curves that were calculated from the east-west component of the 31 October 1935 Helena earthquake. Column (5) results from an average between peaks and valleys. Column (6) results from a smoothing including most of the peaks of the response spectrum curves; it corresponds to an upper average.

Column (7) contains Professor Newmark's amplification factors which correspond to soft rock or firm sediment.

Column (8) contains amplification factors which, multiplied by the maximum ground accelerations, give the results which would be obtained if the maximum ground acceleration had

been reduced by the factor 0.67 as suggested by Professor Newmark when the foundation conditions consist of competent rock.

Column (9) contains the recommended amplification factors. The amplification factors for spectral acceleration shown in Table 2C.3-6(a) are plotted in Figure 2C.3-9. The recommended amplification factors for spectral acceleration are not systematically calculated from any of the amplification factors shown in Table 2C.3-6(a); they are selected by considering all amplification factors with more weight given to those of the Helena earthquake. The recommended amplification factors for spectral accelerations are greater (more conservative) than either of those calculated from the TID 7024 curves, column (3), or those suggested by Professor Newmark for structures built on competent rock, column (8). As requested by the DRL and as stated by the applicant on 26 May 1970, the recommended amplification factors for spectral acceleration are now superseded and were not used in design of the station; instead, the amplification factors for spectral acceleration shown in column (6) for Helena upper average were used for design of the station.

e. Design accelerograms:

The design time-history accelerograms for horizontal vibratory ground motions were developed from the accelerogram of the east-west component of the Helena earthquake. The Helena record was modified so that the resulting accelerograms had the required duration and maximum accelerations and the response spectra computed from these accelerograms are spectra presented in Figures 2C.3-5 and 2C.3-6.

Design accelerogram for Maximum Possible Earthquake (larger earthquake):

The recommended accelerogram for the Maximum Possible (larger) Earthquake was developed by adjusting the Helena record in the following manner:

1. remove the initial 2.4 seconds motion of the Helena record;

2. revise the Helena record from 2.4 seconds to 4.9 seconds with respect to time and change the sign of the accelerations to form a first intermediate accelerogram for the initial 2.5 seconds; increase the duration to 10.0 seconds by adding the 2.4 seconds to 9.9 seconds motion of the Helena record;

3. multiply the time ordinates of the first intermediate accelerogram by a factor of 0.5; the results from 0.0 seconds to 2.0 seconds are used as the initial 2.0 seconds motion for a second intermediate accelerogram. Also, multiply the time ordinates of the first intermediate accelerogram by a factor of 1.4; the results from 2.0 seconds to 10.0 seconds are used as the 2.0 seconds to 10.0 seconds portion of the second intermediate accelerogram;

4. normalize the ordinates of the second intermediate accelerogram to a maximum acceleration of 0.15g; then add to the end of the 10.0 seconds motion five times the 6.0 seconds to 10.0 seconds motion to increase the duration to 30.0 seconds.

Appropriate baseline corrections were made to all accelerograms during each adjustment.

The response spectra of the recommended accelerogram and the corresponding response spectra of the east-west component of the modified Helena earthquake are plotted in Figures

2C.3-10 through 2C.3-14 for a damping ratio ranging from 0.0 to 0.05. These figures show that, in general, the response spectra of the modified Helena accelerogram is larger (more conservative) than the corresponding response spectra shown in Figure 2C.3-5 for frequencies greater than 0.5 cy/sec; frequencies smaller than 0.5 cy/sec are of no concern.

Design accelerogram for Maximum Probable Earthquake (smaller earthquake):

The design accelerogram for the Maximum Probable (smaller) Earthquake was obtained by multiplying the accelerations of the accelerogram of the Maximum Possible (larger) Earthquake by 0.08/0.15.

2C.3.5 Seismic Re-Evaluation

Because of subsequent investigation of the relationship between earthquake intensity and ground acceleration (Trifunac et al. 1975) the NRC staff questioned whether the appropriate ground acceleration for a Modified Mercalli intensity earthquake of VII-VIII should be 0.20g instead of 0.15g. During the Advisory Committee on Reactor Safeguards (ACRS) hearings the ACRS pointed out that the Davis-Besse design criteria were most likely more conservative than the current criteria (i.e., Regulatory Guides 1.60, 1.61, 1.92, etc.) and that possibly the Davis-Besse design for 0.15g was equivalent to a current design for 0.20g. A condition was placed in the Davis-Besse Operating License requiring the licensee to perform a seismic re-evaluation to demonstrate that the Davis-Besse design provided adequate margin for a 0.20g Maximum Possible Earthquake, using current criteria.

During the first fuel cycle, a seismic re-evaluation was performed following NRC staff guidelines. The re-evaluation, using a Maximum Possible Earthquake ground acceleration of 0.20g and current criteria determined that there were still sufficient margins available in systems required for safe shutdown of the unit as well as those systems required for continued shutdown heat removal.

On August 27, 1980, the Commission issued Amendment 30 to the Davis-Besse operating license, deleting the license condition requiring the seismic re-evaluation.

On May 31, 1983, the Commission issued a safety evaluation of the seismic re-evaluation which concluded that there is sufficient conservatism and margin in the piping systems, components and supports at Davis-Besse to ensure safe shutdown and continued heat removal in the event of an earthquake having a ground acceleration of 0.20g.

The Davis-Besse Nuclear Power Station (DBNPS) continues to use the seismic design parameters described in Section 2C.3.4.

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TABLE 2C.3-1

List of Historically Reported Earthquakes of Epicentral Intensity As Defined in Figure 2C.3-2

Year	Date	Location	Distance from Site, mi	Lat N°	Long W°	Intensity MM	Felt Area in 1000 mi.²	Richter Magnitude Instru-mental	Richter Magnitude Based On Felt Area (H)	Richter Magnitude Based On Felt Area (N)	Intensity at Site MM
1804	24 Aug	Ft Dearborn, Ill	240	42.0	87.8	-	30		4.2	4.4	0
1811	16 Dec	New Madrid, Mo	470	36.6	89.6	X-XI	2000		7.9	7.2	V
1812	23 Jan	New Madrid, Mo	470	36.6	89.6	X	2000		7.9	7.1	V
1812	7 Feb	New Madrid, Mo	470	36.6	89.6	XI	2000		7.9	7.4	V
1827	6 Aug	New Albany, Ind	265	38.3	85.8	VI	?				
1827	7 Aug	New Albany, Ind	265	38.3	85.8	VI	?				
1857	23 Oct	Near Buffalo, NY	250	43.2	78.6	VI	8		3.2	4.1	
1872	6 Feb	Wenona, Mich	135	43.5	83.8	IV	local		3.0		
1873	6 July	15 Mi ±W Of Welland, Ont	200	43.0	79.5	VI	30		3.6	4.4	
1875	18 June	Anna, Ohio	100	40.2	84.0	VII	40		4.4	4.5	
1882	9 Feb	Near Anna, Ohio	100	?	?	V	?				
1883	4 Feb	Southwestern Michigan	140	42.3	85.6	VI	8		3.4	4.1	
1884	19 Sept	Near Lima, Ohio	80	40.7	84.1	V	125		5.3	4.8	
1886	31 Aug	Charleston, SC	620	32.9	80.0	X	2000		7.1		III (Wo)
1887	6 Feb	Vincennes, Ind	300	38.7	87.5	V-VI	75		4.8	4.7	
1897	31 May	Giles County, Va	315	37.3	80.7	VIII	280		5.2	5.1	≤ III
1899	29 Apr	Southwest Indiana	290	38.5	87.0	VI-VII(6.5)	40		4.4	4.5	
1901	17 May	Near Creola, Ohio	155	39.3	82.5	V	7		3.4	4.0	
1906	27 June	Fairport, Ohio	80	41.4	81.6	V	0.4		3.0	3.4	
1909	26 May	Near Rockford, Ill	305	42.5	89.0	VII	500		6.6	5.3	
1909	27 Sept	Near Linton, Ind	300	39.0	87.7	VII	30		4.2	4.4	
1912	2 Jan	Near Joliet, Ill	280	41.5	88.5	VI	40		4.4	4.5	
1925	1 Mar	St Lawrence Valley	770	47.6	70.1	IX-X (9.5)	>1000	7 (s)	>6.4	>5.5	III (s)
1926	5 Nov	NW Of Pomeroy, Ohio	180	39.1	82.1	VI-VIII (6.5)	0.35		3.0	3.4	
1928	9 Sept	Cleveland, Ohio	55	41.5	82.0	V	1.5		3.0	3.7	0 (c)
1929	8 Mar	Bellefontaine, Ohio	100	40.4	84.2	V	5		3.3	4.0	
1929	12 Aug	Attica, NY	270	42.9	78.3	VIII	100	5.8 (s)	4.4	4.8	0
1930	20 Sept	Anna, Ohio	100	40.4	84.2	VI	?				
1930	30 Sept	Anna, Ohio	100	40.4	84.2	VIII	?				

TABLE 2C.3-1 (Continued)

List of Historically Reported Earthquakes of Epicentral Intensity As Defined in Figure 2C.3-2

								Richter Magnitude			
Year	Date	Location	Distance from Site, mi	Lat N°	Long W°	Intensity MM	Felt Area in 1000 mi.²	Instru-mental	Based On Felt Area (H)	Based On Felt Area (N)	Intensity at Site MM
1931	10 June	Malinta, Ohio	50	41.3	84.0	V	?				
1931	20 Sept	Anna, Ohio	100	40.4	84.2	VII	40		4.4	4.5	0 (C)
1933	28 May	Maysville, Ky	200	38.6	83.7	V	0.6		3.0	3.0	
1933	20 Sept	Anna, Ohio	100	?	?	VI	?				
1934	29 Oct	Erie, Pa	150	42.0	80.2	V	local				
1937	2 Mar	Near Anna, Ohio	100	40.7	84.0	VI-VII (6.5)	90		5.0	4.7	III
1937	3 Mar	Near Anna, Ohio	100	40.7	84.0	V	?				
1937	8 Mar	Near Anna, Ohio	100	40.4	84.1	VII-VIII (7.5)	150	5.5 (G)	5.5	4.9	IV
1943	8 Mar	Lake Erie	115	42.2	80.9	V	40	5.5 (S) 3.7-4.3 (W)	4.4	4.5	<III
1947	9 Aug	South Central Michigan	100	42.0	85.0	VI	50	4.9 (W)	4.5	4.6	III
1952	20 June	Near Malta, Ohio	135	39.7	82.1	VI	10		3.5	4.1	<III
1955	26 May	Cleveland, Ohio	75	41.5	81.7	V	local		3.0		<V
1955	28 June	Cleveland, Ohio	75	41.5	81.7	IV-V	local		3.0		0
1956	27 Jan	Lima, Ohio	80	40.5	84.0	V	?	4.4 (S) 3.1 (W)			III
1957	29 June	9 Mi SSE of London, Ont	130	42.9	81.3	V	?	4.2 (S)			
1958	1 May	Near Cleveland, Ohio	80	41.5	81.6	IV-V	local	2.5 (W)	3.0		
1961	22 Feb	Near Fostoria, Ohio	30	41.2	83.4	V	?				III
1966	1 Jan	Near Attica, NY	225	42.8	78.2	VI	3.5	4.7 (B)	3.1	3.9	
1967	7 Apr	Sugar Grove, Ohio	145	39.6	82.5	V	4	4.2 (C&B)	3.2	3.9	
1967	13 June	Near Attica, NY	250	42.9	78.2	VI	3	3.9			
1968	9 Nov	Near Marion, Ill	370	38.0	88.8	VII	580	5.3			
1969	19 Nov	Southern W.V.	200	37.4	81.0	VI	100		4.4	4.8	

Notes: 1. Bradley, personal communications; (C) USC&GS; (G) Gutenberg et al, 1954; (H) Housner, 1969; (N) Nuttli et al, 1974; (S) Smith, 1962; Smith, 1966: (W) Walter, personal communication; (WO) Wollard, 1958.

2. Table is complete through August, 1976.

TABLE 2C.3-2

List of Historically Reported Earthquakes within Approximately 100 Miles of Site

				Epicenter Location				Richter Magnitude			
Year	Date	Location	Distance from Site, mi	Lat N°	Long W°	Intensity MM	Felt Area in 1000 mi.²	Instru-mental	Based on Felt Area (H)	Based on Felt Area (N)	Intensity at Site MM
1872	23 July	Near Lorain, Ohio	60	41.5	81.8	III	?				
1875	18 June	Anna, Ohio	100	40.2	84.0	VI-III (6.5)	46.5		4.4	4.5	
1876	June	Anna, Ohio	100	40.2	84.0	IV	?				
1877	17 Aug	Redford, Mich	50	42.3	83.3	IV	0.2		3.0	3.3	
1882	9 Feb	Near Anna, Ohio	80	40.4	84.2	V	?				
1884	19 Sept	Near Lima, Ohio	80	40.7	84.1	V	125		5.3	4.8	
1884	23 Dec	Anna, Ohio	100	40.4	84.2	III	?				
1896	15 Mar	Sidney, Ohio	105	40.3	84.2	IV	?				
1906	27 June	Fairport, Ohio	80	41.4	81.6	V	0.4		3.0	3.4	
1926	28 Oct	Toledo, Ohio	25	41.6	83.6	III	?				II
1927	16 Feb	Mansfield, Ohio	60	40.9	82.6	IV	?				
1928	9 Sept	Cleveland, Ohio	55	41.5	82.0	V	1.5		3.0	3.7	0 (C)
1928	27 Oct	Jackson Ctr, Ohio	90	40.4	84.0	III	?				
1929	8 Mar	Bellefontaine, Ohio	100	40.4	84.2	V	6.2		3.3	4.0	
1930	26 June	Lima, Ohio	80	40.5	84.0	IV	?				
1930	27 June	Lima, Ohio	80	40.5	84.0	IV	?				
1930	11 July	Marion, Ohio	60	40.7	83.2	IV	?				
1930	20 Sept	Anna, Ohio	100	40.4	84.2	VI	?				
1930	29 Sept	Sidney, Ohio	105	40.4	84.2	III	?				
1930	30 Sept	Anna, Ohio	100	40.4	84.2	VII					
1930	Oct	Anna, Ohio	100	40.4	84.2	III-IV	?				
1930	20 Nov	Oxbow, Mich	75	42.6	83.0	III	?				
1931	21 Mar	Sidney, Ohio	105	40.4	84.2	III	?				0 (C)
1931	31 Mar	Jackson Ctr, Ohio	90	40.4	84.0	III	?				
1931	10 June	Malinta, Ohio	50	41.3	84.0	V	?				
1931	20 Sept	Anna, Ohio	100	40.4	84.2	VII	40		4.4	4.5	0 (C)
1931	8 Oct	Anna, Ohio	100	40.4	84.2	III	?				

TABLE 2C.3-2 (Continued)

List of Historically Reported Earthquakes within Approximately 100 Miles of Site

Year	Date	Location	Distance from Site, mi	Epicenter Location Lat N°	Epicenter Location Long W°	Intensity MM	Felt Area in 1000 mi.²	Richter Magnitude Instru-mental	Richter Magnitude Based On Felt Area (H)	Richter Magnitude Based On Felt Area (N)	Intensity at Site MM
1932	21 Jan	Akron, Ohio	80	41.1	81.6	IV	?				
1933	22 Feb	Sidney, Ohio	105	40.3	84.2	IV	?				
1933	20 Sept	Anna, Ohio	100	?	?	VI					
1936	31 Jan	Tiffin, Ohio	30	41.1	83.2	II	?				
1937	2 Mar	Near Anna, Ohio	100	40.7	84.0	VII	90		5.0	4.7	III
1937	3 Mar	Near Anna, Ohio	100	40.7	84.0	V	?				
1937	3 Mar	Near Anna, Ohio	100	40.7	84.0	III					
1937	8 Mar	Near Anna, Ohio	100	40.4	84.1	VII-VIII (7.5)	150	5.5 (G)	5.5	4.9	IV
1937	23 Apr	Near Anna, Ohio	100	40.7	84.0	III	?				
1937	27 Apr	Anna, Ohio	100	40.7	84.0	III	?				
1937	2 May	Anna, Ohio	100	40.7	84.0	IV	?				
1938	13 May	Detroit, Mich	50	42.3	83.0	II	?				
1939	18 Mar	Jackson Ctr, Ohio	90	40.4	84.3	II	?				
1939	18 Mar	Jackson Ctr, Ohio	90	40.4	84.1	III-IV	?				
1939	17 June	Anna, Ohio	100	40.3	84.0	IV	?				
1939	9 July	Anna, Ohio	100	40.3	84.0	II	?				
1940	31 May	W. Akron, Ohio	85	41.1	81.5	II	?				
1940	16 June	Mankin, Ohio	65	40.9	82.3	III	?				
1940	28 July	Mankin, Ohio	65	40.9	82.3	II	?				
1940	15 Aug	Mankin, Ohio	65	40.9	82.3	II	?				
1940	19 Aug	Mankin, Ohio	65	40.9	82.3	II	?				
1944	13 Nov	Near Anna, Ohio	100	40.4	84.4	III	?				
1947	9 Aug	South Central Mich	100	42.0	85.0	VI	50		4.5	4.6	III
1948	18 Jan	Near Toledo, Ohio	25	?	?	III	?				

TABLE 2C.3-2 (Continued)

List of Historically Reported Earthquakes within Approximately 100 Miles of Site

				Epicenter Location				Richter Magnitude			
Year	Date	Location	Distance from Site, mi	Lat N°	Long W°	Intensity MM	Felt Area in 1000 mi.²	Instru-mental	Based On Felt Area (H)	Based On Felt Area (N)	Intensity at Site MM
1951	3 Dec	Willoughby, Ohio	80	41.6	81.4	IV	?				
1951	7 Dec	Willoughby, Ohio	80	41.6	81.4	II	?				
1951	21 Dec	Willoughby, Ohio	80	41.6	81.4	II	?				
1953	11 June	Toledo, Ohio	25	41.6	83.6	IV	?				
1955	26 May	Cleveland, Ohio	75	41.5	81.7	V	local				<V
1955	28 June	Cleveland, Ohio	75	41.5	81.7	IV-V	local				0
1956	27 Jan	Lima, Ohio	80	40.5	84.2	V	?	4.4 (S) 3.1 (W)			
1958	1 May	Near Cleveland, Ohio	80	41.5	81.6	IV-V	local	2.5 (W)			
1961	22 Feb	Near Fostoria, Ohio	30	41.2	83.4	V	?				III
1967	2 Feb	Lansing, Mich	100	42.8	84.3	IV					
1968	26 July	Anna, Ohio	95	?	?	≥ III	0.5	3.0 (Wi)	3.0	3.5	
1968	31 Oct	Port Huron, Ohio	97	42.9	82.4	≥ IV	10.4		3.5	4.1	
1974	29 Sept	Near Bowling Green, Ohio	30	41.2	83.4	II-III	3.0				

Notes: 1. (C) USC&GS; (G) Gutenberg et al, 1964, (H) Housner, (1969); (N) Nuttli et al, (1974), (S) Smith, 1966, (W) Walter, personal communication; (Wi) Willis and Wilson, 1970.

2. Table is complete through August 1976.

TABLE 2C.3-3

List of Historically Reported Earthquakes in Anna Vicinity

			Epicenter				Richter Magnitude			
Year	Date	Location	Lat N°	Long W°	Intensity MM	Remarks	Instru-mental	Based On Felt Area (H)	Based On Felt Area (N)	Intensity at Site MM
1875	18 June	Anna, Ohio	40.2	84.0	VII	Bradley, Bennett 1965 Docekal		4.4	4.5	
1876	June	Anna, Ohio	40.2	84.0	IV					
1882	9 Feb	Near Anna, Ohio	40.4	84.2	V	Docekal				
1884	19 Sept	Near Lima, Ohio	40.7	84.1	V	Docekal		5.3	4.5	
1884	23 Dec	Anna, Ohio	40.4	84.2	III	Bradley, Docekal				
1889	Sept	Anna, Ohio	40.4	84.2	III					
1892	Summer	Anna, Ohio	40.4	84.2	---					
1896	15 Mar	Sidney, Ohio	40.3	84.2	IV	Bradley, Docekal				
1914	----	Anna, Ohio	40.4	84.2	III					
1925	Oct	Anna, Ohio	40.4	84.2	III					
1928	27 Oct	Jackson Ctr, Ohio	40.4	84.0	III	Bradley, Docekal				
1929	8 Mar	Bellefontaine, Ohio	40.4	84.2	V	Docekal		3.3	3.4	
1930	26 June	Lima, Ohio	40.5	84.0	IV	Docekal				
1930	27 June	Lima, Ohio	40.5	84.0	IV	Smith, Docekal				
1930	11 July	Marion, Ohio	40.7	83.2	IV					
1930	20 Sept	Anna, Ohio	40.	84.2	VI	Westland, Docekal				
1930	29 Sept	Sidney, Ohio	40.4	84.2	III	Docekal				
1930	30 Sept	Anna, Ohio	40.4	84.2	VII	Bradley, Docekal				
1930	Oct	Anna, Ohio	40.4	84.2	III-IV					
1931	21 Mar	Sidney, Ohio	40.4	84.2	III	82.4° may be a misprint in Smith				0
1931	31 Mar	Jackson Ctr, Ohio	40.4	84.0	III	Bradley, Docekal				
1931	20 Sept	Anna, Ohio	40.4	84.2	VII	Docekal		4.4	4.5	0
1931	8 Oct	Anna, Ohio	40.4	84.2	III	Bradley, Docekal				

TABLE 2C.3-3 (Continued)

List of Historically Reported Earthquakes in Anna Vicinity

			Epicenter				Richter Magnitude			
Year	Date	Location	Lat N°	Long W°	Intensity MM	Remarks	Instru-mental	Based on Felt Area (H)	Based on Felt Area (N)	Intensity at Site MM
1933	22 Feb	Sidney, Ohio	40.3	84.2	IV	Westland writes MM VI, USC&GS does not list; 82.4° may be misprint in Smith, Docekal				
1933	20 Sept	Anna, Ohio	?	?	VI	Westland, Docekal				
1937	2 Mar	Anna, Ohio	40.7	84.0	VII	Docekal		5.0	4.7	III
1937	3 Mar	Anna, Ohio	40.7	84.0	V	Docekal				
1937	3 Mar	Anna, Ohio	40.7	84.0	III					
1937	8 Mar	Anna, Ohio	40.4	84.1 (7.5)	VII-VIII	Docekal	5.5 (G)	5.5	4.9	IV
1937	23 Apr	Anna, Ohio	40.7	84.0	III	Westland (MM after Bradley)				
1937	27 Apr	Anna, Ohio	40.7	84.0	III	Bradley, Docekal				
1937	2 May	Anna, Ohio	40.7	84.0	IV	Bradley, Docekal				
1939	18 Mar	Jackson Ctr, Ohio	40.4	84.3	II	Bradley, Docekal				
1939	18 Mar	Jackson Ctr, Ohio	40.4	84.1	III-IV					
1939	17 June	Anna, Ohio	40.3	84.0	IV					
1939	9 July	Anna, Ohio	40.3	84.0	II					
1944	13 Nov	Near Anna, Ohio	40.4	84.4	III	Bradley, Docekal				
1956	27 Jan	Lima, Ohio	40.5	84.2	V	MM after Bradley Docekal	4.4 (S) 3.1 (W)			
1968	26 July	Anna, Ohio	?	?	III	Willis and Wilson Docekal	3.0 (Wi)			

Notes: 1. All earthquakes are from the Dominion Observatory of Canada (Smith, 1962; Smith, 1966); USC&GC (Eppley, 1965); Westland (Westland et al, 1940); and Bradley (Bradley et al 1965). Where Westland and/or Bradley are listed remarks, neither Eppley nor Smith list the earthquakes. (G) Gutenberg et al, 1954; (H) Housner, 1969; (N) Nuttli et al, 1974; (S) Smith, 1966; Walter, personal communication; (Wi) Willis and Wilson, 1970.

2. Table is complete through August, 1976; Bradley, personal communication.

TABLE 2C.3-4

Recommended Parameters for the Design Earthquakes

I. Horizontal Vibratory Ground Motions

A. Maximum Possible Earthquake (larger earthquake)

Maximum ground acceleration:	0.15 gravity
Maximum ground velocity:	5 in/sec
Maximum ground displacement:	3.33 in.
Total duration:	30 sec

Time-history accelerograms. The accelerograms is recommended in Section III.D.4.e.

B. Maximum Probable Earthquake (smaller earthquake)

Maximum ground acceleration:	0.08 gravity
Maximum ground velocity:	2.67 in/sec
Maximum ground displacement:	1.78 in.
Total duration:	30 sec

Time-history accelerograms - The accelerograms is recommended in Section III.D.4.e.

II. Vertical Vibratory Ground Motions

Maximum Possible (larger) Earthquake and Maximum Probable (smaller) Earthquake

Vertical vibratory ground motions are 2/3 of the respective maximum horizontal vibratory ground motions.

TABLE 2C.3-5

Comparison of Several Earthquakes

	Maximum Possible Earthquake (larger earthquake) for Davis-Besse Station	Helena 31 October 1935	Golden Gate 22 March 1957	Taft 21 July 1952	El Centro 18 May 1940
Epicentral Intensity:	Medium VII (7.5)	VIII	VII	XI	X
Richter Magnitude:	6	6	5.5	7.7	7.1
Recording Station:					
Subsurface Conditions:	15 ft glacial deposits. Bedded dolomite and shale. Silurian.	Few feet of lake bed deposit. Marsh shale and Helena limestone. Tertiary and Paleozoic.	Jurassic sandstone.	50 ft alluvium (clayey sand). Very soft sedimentary rock.	More than 1000 ft of alluvium.
Representative Seismic Velocities (2):					
P-wave, ft/sec	12500	15000			
S-wave, ft/sec	6500	8000	1700/4500	2000	900-1500
Type of Structure:	To be used for design of reinforced concrete and steel structure built into rock	Federal building 4-stories reinforced concrete and steel columns. Well-built on rock. Not damaged by earthquake.	Floor of 10 ft by 10 ft reinforced concrete pump house structure built on rock.	In a tunnel between two buildings built on soil.	Concrete basement built on soil.
Type of Foundation Material of Support of Seismograph:	Rock motion is predicted	Rock	Rock	Soil	Soil
Epicentral Distance, mi:	Local	2-4	8	30	27
Maximum Ground Motion;					
acceleration/-g	0.15	(1) 0.149	.13	.18	.33
velocity, in/s	5	(1) 6.45	1.85	5.82	17.5
displacement, in.	3.33	(1) 3.65	0.92	6.71	11.9
Predominant Period, s	0.25	0.25	0.22	0.35	0.5

Notes: (1) Calculated from the accelerogram selected for Davis-Besse Station.
(2) Of materials within approximately 150 feet below seismograph.

TABLE 2C.3-6

Study of Response Spectra

(a) Amplification Factors for Spectral Acceleration

High Frequency Range; eg f = 5 cy/sec

(1) Damping Ratio	(2) E-E	(3) Housner TID7024	(4) Amin U of Ill	(5) Helena Avg	(6) Helena Upper Avg	(7) Newmark	(8) Newmark x0.67	(9) Recom-mendations for Davis-Besse Station
0	8.0	5.7	6.6	3.8	5.3	6.4	4.3	5.7
0.005	4.8	4.7		3.6	4.7	5.8	3.8	4.4
0.01	3.8	3.7		3.5	4.0	5.2	3.5	3.8
0.02	2.9	2.3	3.0	2.8	3.3	4.3	2.9	3.0
0.05	2.0	1.5	2.0	2.2	2.5	2.6	1.7	2.2
0.10	1.0	1.3	1.7	1.7	1.8	1.5	1.0	1.7

see note 1

(b) Amplification Factors for Spectral Velocity

Medium Frequency Range; eg f = 1 cy/sec

(1) Damping Ratio	(2) E-E	(3) Housner TID7024	(4) Amin U of Ill	(5) Helena Avg	(6) Helena Upper Avg	(7) Newmark	(8) Newmark x0.67	(9) Recom-mendations for Davis-Besse Station
0	3.8	3.8	3.7	2.8	3.0	4.0		3.8
0.005	3.4	3.4		2.5	2.7	3.6		3.2
0.01	3.0	2.7		2.5	2.7	3.2		2.7
0.02	2.4	2.4	2.2	2.3	2.4	2.8		2.4
0.05	1.9	2.0	2.1	2.1	2.2	1.9		2.0
0.10	1.0	1.7	2.0	1.7	1.9	1.3		1.5

(c) Amplification Factors for Spectral Displacement

Low Frequency Range; eg f = 0.2 cy/sec

(1) Damping Ratio	(2) E-E	(3) Housner TID7024	(4) Amin U of Ill	(5) Helena Avg	(6) Helena Upper Avg	(7) Newmark	(8) Newmark x0.67	(9) Recom-mendations for Davis-Besse Station
0	2.2				1.3	2.5		2.5
0.005	2.1				1.3	2.2		2.2
0.01	2.0				1.3	2.0		2.0
0.02	1.8				1.2	1.8		1.5
0.05	1.6				1.1	1.4		1.3
0.10	1.0				1.0	1.1		1.0

Note 1: As requested by DRL, amplification factors for spectral acceleration for “Helena Upper Ave” are used for Davis-Besse Station

2C.4.0 Subsurface Conditions

2C.4.1 Introduction

Preconstruction geotechnical investigations were made to obtain data concerning soil, bedrock, and groundwater conditions. These investigations were made by Woodward-Moorhouse & Associates, Inc. (WMAI), except for a preliminaiy investigation made by Toledo Testing Laboratory of Toledo, Ohio. Results of the preliminary investigation are included in this Appendix.

During construction, additional geotechnical investigations were made to obtain data necessary to satisfy quality assurance requirements and to provide geotechnical design parameters not obtained during preconstruction investigations. The following geotechnical investigations were made in the station area[1]: (1) a Bedrock Verification Program; (2) a Groundwater Monitoring Program; and (3) a Bedrock Socket-Concrete Bond Strength Testing Program. In addition to these programs, geotechnical investigations were made during construction in borrow areas, the Switchyard area, the Concrete Batch plant area, and the Cooling Tower area.

This Appendix presents the generalized subsurface conditions at the site and the specific subsurface conditions in the station area considering the results of all geotechnical investigations made at the site.

2C.4.2 Scope of Preconstruction Geotechnical Investigations

1. Field Exploration

a. General:

The preconstruction field exploration consisted of borings, soil probes, rock probes, a seismic survey, a test excavation, a groundwater study, and bedrock permeability tests.

b. Borings:

Fifty-seven (57) borings were drilled to obtain soil and bedrock samples. The locations of the borings are shown in Figures 2C.4-1 through 2C.4-3. Standard penetration determinations were made at intervals in the soil deposits and samples were obtained. NX2-dia rock cores were obtained in bedrock. The borings penetrated to depths ranging from approximately 30 ft to 200 ft below existing ground surface.

c. Soil probes:

One hundred thirty-two (132) unsampled soil probes were drilled to refusal to determine the elevation of the bedrock surface. The locations of the soil probes were essentially confined to the surface depression area located approximately 400 ft south of the station area. The locations of 127 of the 132 soil probes are shown in Figures 2C.4-1 through 2C.4-3. The locations of five soil probes are not shown. These soil probes are located along the north-south oriented probe line south of Area A (see Figure 2C.4-1).

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[1] Station area in this Appendix refers to the 570-ft by 750-ft area indicated in Figure 2C.4-1.

[2] The prefix NX- specifies the size of rock cores obtained during drilling (NX-sized rock core is 2-1/8 in. dia).

d. Rock probes:

Eighty-eight (88) rock probes were drilled to explore for the possible existence of cavities in the upper 25 ft to 35 ft of bedrock in the station area. The locations of the rock probes are shown in Figure 2C.4-2.

e. Seismic survey:

One hundred forty (140) seismic recordings were made to determine the elevation of bedrock between boring and probe locations and to determine whether anomalous subsurface conditions were present in the station area. The locations of seismic survey lines are shown in Figures 2C.4-1 through 2C.4-3. Seismic measurements included determinations of both the compression wave (P-wave) and shear wave (S-wave) velocity of the bedrock.

In addition, two lines of four broadside seismic refraction measurements were made in the surface depression area to assist in determination of the extent of the solution activity found in this area.

f. Test excavation:

Three sections of a test excavation were made in the surface depression area to study the characteristics of solution activity. The location, horizontal extent, and depth of each section are shown in Figure 2C.4-3.

g. Groundwater study:

Regional and site area groundwater conditions were studied. The regional groundwater study consisted of a literature review, interviews with local residents and representatives of the Ohio Department of Natural Resources, and a study of 32 logs of local wells. The site groundwater study consisted of installation and intermittent monitoring of 18 piezometers within the site area boundaries. Monitoring began in July 1968. Locations of piezometers are shown in Figures 2C.4-1 and 2C.4-2.

A discussion of regional and site groundwater conditions is presented in Section 2.4.13 of the USAR. A discussion of site groundwater conditions is presented in this section.

h. Bedrock permeability tests:

Twenty-nine (29) pump-in permeability tests were made at various depths in the bedrock portion of six borings (B2-3, B2-5, B2-7, B3-1, B3-2, and B3-5); pump-out permeability tests were made in bedrock portion of boring B2-10 and rock probe No. 53 (coordinate location of rock probe 53 is N10324, E10174), and in the test excavation made in the surface depression area.

2. Laboratory Investigation:

a. General:

The preconstruction laboratory investigation consisted of classification, index property, compressibility, strength, and ultrasonic velocity tests made on samples of soil and bedrock.

b. Soil testing:

Soil samples were visually classified in the field, and this visual classification was reviewed by laboratory personnel. The following index property tests were made to assist in classification of soil samples: 268 water contents, 75 Atterberg limits, 34 grain-size analyses, and 31 unit weights.

Eight consolidation tests were made. Two were made on undisturbed samples of glaciolacustrine deposit and six were made on undisturbed samples of till deposit.

One hundred ninty-six (196) unconfined compression tests and 13 unconsolidated-undrained triaxial compression tests were made on samples of glaciolacustrine and till deposit. Three dynamic triaxial tests were made on samples of till deposit.

c. Bedrock testing:

Bedrock core samples were visually classified and Rock Quality Designation (RQD) was determined for each core run. Eleven (11) 2-ft-long cores, representative of the major bedrock units at the site, were identified by Dr. L. F. Rooney, Head of the Industrial Minerals Section, Indiana Geological Survey. Chemical analyses were made on seven cord samples.

Index property tests consisted of two water content and four unit weight determinations. Eighteen (18) unconfined compression tests were made on selected cord samples.

Eleven (11) dynamic triaxial tests were made on core samples of the two major dolonitic bedrock types.

2C.4.3 Scope of Geotechnical Investigations During Construction

1. General:

During construction, a Bedrock Verifications Program was implemented to determine the degree of significant solution activity, if any, in bedrock beneath major structures in the station area; a Groundwater Monitoring Program was implemented to determine if dewatering operations during construction at the site were causing significant solution activity; and a Load Test Program was implemented to determine the allowable bedrock socket-concrete bond strength for use in the design of piers supporting Turbine and Auxiliary Building columns. In addition to these investigation programs, geotechnical investigations were also made in the Concrete Batch Plant area, borrow area C, Switchyard area, and Cooling Tower area to obtain specific geotechnical design data.

2. Bedrock Verification Program:

The purpose, requirements, and generalized scope of the Bedrock Verification Program and a discussion of the results obtained are presented in Section 2C.5 of this Appendix.

3. Groundwater Monitoring Program:

The purpose, requirements, and generalized scope of the Groundwater Monitoring Program and a discussion of the results obtained are presented in Section 2C.5 of this Appendix.

4. Bedrock Socket-Concrete Bond Strength Tests:

Six bedrock socket-concrete bond strength tests were made to determine an allowable bedrock socket-concrete bond strength for use in the design of pier footings at the site. The six tests consisted of two push[1] tests, two "shallow" pull[2] tests, and two "deep" pull tests. The location of the six tests is shown in Figure 2C.4-4. Conclusions based on results of the tests are presented in this section.

5. Miscellaneous Geotechnical Investigations:

Figure 2C.4-1 shows the location of borings and probes made during the geotechnical investigation in the Concrete Batch Plant area, borrow area C, Switchyard area, and Cooling Tower area. Results of borings and probes made during these investigations were used in the determination of site subsurface conditions. Conclusions based on these results are presented in this section.

2C.4.4 Description and Static Physical Properties of Soil and Bedrock

1. General:

Surficial soil deposits in the site area consist of glaciolacustrine deposit in the western one-half and of organic and beach deposits in the eastern one-half.

Existing site grades prior to construction ranged from an average of El. 570 in the marsh area east of the station area to an average of El. 574 in the station area. During construction, site grades in the station area were raised to approximately El. 583 by placement and compaction of fill. Site grades in the marsh area remained at El. 570, except along the Intake Canal alignment where two dikes were constructed to approximately El. 579. Along the lake front, site grades remained essentially unchanged. Figure 2C.4-5 shows distribution of surficial deposits in the site area after construction.

Three generalized geologic profiles were prepared which present the distribution and physical properties of the soil deposits and bedrock; see Figures 2C.4-6 through 2C.4-8. These profiles show that the site is underlain by two glacial soil deposits which overlie a dolomitic bedrock formation. The glacial deposits consist of an upper glaciolacustrine deposit and a lower till deposit. Marsh organic and beach deposits overlie the glacial deposits in the eastern portion of the site. The total thickness of the soil deposits ranges from approximately 14 ft in the station area to approximately 25 ft at the lake front along the Intake Canal alignment.

The description, distribution, and physical characteristics of the soil deposits and of the major bedrock units are presented in the following text.

2. Glaciolacustrine Deposit:

The glaciolacustrine deposit consists of stiff fissured, desiccated, medium plastic, mottled, gray and brown silty clay. Its thickness in the station area was generally found to range from 6 ft to 10 ft. Its thickness along the canal alignments was found to range from 6 ft to 12 ft. The upper

[1] Push tests refer to tests made by applying a compression load to the test caisson.
[2] Pull tests refer to tests made by applying a tensile load to the test caisson.

0.5 ft to 1.5 ft of this deposit contains roots and organic material and is referred to as topsoil. In the lower few feet, some stratification is evident.

Representative values of index and strength properties of the glaciolacustrine deposit are: natural water content, 24%; liquid limit, 51; plastic limit, 23; unit weight, 125 lb/cu ft; unconfined compressive strength, 3.5 t/sq ft; and standard penetration resistance, 12 lb/ft. It is estimated, on the basis of plasticity characteristics, that the coefficient of permeability of the glaciolacustrine deposit is less than $1x10^{-6}$ cm/sec.

Consolidation tests made on samples of a glaciolacustrine deposit indicate the deposit was consolidated under a maximum past effective pressure of 4 k/sq ft to 12 k/sq ft. The maximum existing effective pressure is approximately 1 k/sq ft. The compression index C ranges from 0.01 to 0.04 and the coefficient of consolidation C was found to be $0.5x10^{-2}$ sq cm/sec for the loading range 2-8 k/sq ft.

The index, strength, and consolidation data indicate that the glaciolacustrine deposit has a low compressibility.

3. Till Deposit:

The till deposit consists of a hard fissured, desiccated, low to medium plastic, gray to brown silty sandy clay with less than 10% gravel, except for a 1- to 3-ft zone overlying the bedrock surface which consists of silty sandy clay boulders. Discontinuous lenses of silty sand are occasionally encountered in this 1- to 3-ft zone. The thickness of the till deposit in the station area was found to range from 6 ft to 10 ft. Its thickness along the Intake Canal alignment was found to range from 6 ft to 12 ft. The contact between the till deposit and the overlying glaciolacustrine deposit is not always sharp. In several borings, there appears to be a transition zone. It is estimated, on the basis of plasticity characteristics, that the coefficient of permeability of the till deposit is less than $1x10^{-6}$ cm/sec.

Representative values of index and strength properties of the till deposit are: natural water content, 15%; liquid limit, 33; plastic limit, 17; unit weight, 132 lb/cu ft; unconfined compressive strength, 8 t/sq ft; and standard penetration resistance, 40 lb/ft.

The consolidation tests made on samples of the till deposit indicate the deposit was consolidated under a maximum past effective pressure of 10 k/sq ft to 50 k/sq ft. The compression index, C_c, ranges from 0.06 to 0.12, the recompression index C_r ranges from 0.01 to 0.02, and the coefficient of consolidation C_v was found to be $1x10^{-2}$ sq cm/sec for the loading range 2-8 k/sq ft.

The index, strength, and consolidation data indicate that the till deposit has a very low compressibility.

4. Organic Deposit:

The organic deposit consists of decayed organic matter (peat) and gray to black clay with less than 10% sand. The organic deposit was encountered in the marsh below water level. Along the Intake Canal alignment, its thickness ranged from 0 ft to 3 ft. At the lake front, a thin layer (0.5 ft) of this deposit was encountered below the sand deposit.

Index and strength properties of the organic deposit vary widely: natural water content, 62% to 277%; liquid limit, 81 to 95; plastic limit, 36 to 47; unit weight, 75 to 100 lb/cu ft; and unconfined

compressive strength, 0.1 to 0.25 t/sq ft. The index and strength data indicate that the organic deposit has a high compressibility.

5. Sand Deposit:

The sand deposit consists of a poorly graded to well graded silty medium to fine sand with less than 10% gravel. The sand deposit was encountered along the Lake Erie shoreline: see Figure 2C.4-5. The thickness of the sand deposit in the borings was approximately 10 ft. Representative value of standard penetration resistance was 5 lb/ft, indicating the deposit has a low relative density.

6. Bedrock Formation:

a. General:

The bedrock formation penetrated by the borings is the Tymochtee formation which consists of argillaceous dolomite containing interbedded gypsum, anhydrite, and shale strata. Borings drilled below El. 460 penetrated through the Tymochtee formation into the upper portions of the Greenfield formation. Figures 2C.4-6 through 2C.4-8 present generalized geologic profiles showing the occurrence and properties of bedrock in the station area. Figure 2C.4-1 shows the inferred contours of the top of the bedrock surface in the site area based on results of borings and probes made during preconstruction and construction subsurface investigations. Bedrock surface elevations ranged from approximately El. 565 in borrow area C to El. 545 off-shore along the Lake Erie shoreline.

b. Dolomite:

The argillaceous dolomite strata can be divided into two rock types: a massive dolomite and a laminated dolomite. The laminated dolomite is referred to as bedded dolomite in this Appendix.

The massive dolomite occurs in an 8 ft to 10 ft thick stratum, the top of which is located approximately 10 ft below the existing bedrock surface. It is a medium hard to hard, gray to buff, fine grained, argillaceous dolomite containing anhydrite vugs in the upper 4 ft to 6 ft. In the station area, core obtained from the borings indicated the massive dolomite to be sound and unweathered. Typical rock core recovery ranged between 95 and 100 percent with the average being 98 percent.

The bedded dolomite occurs both above and below the massive dolomite stratum. It is a medium hard, gray to buff, argillaceous dolomite with frequent laminae of gypsum, anhydrite, and shale.

The bedded dolomite above approximately El. 505 contains approximately 20 percent gypsum, 5 percent anhydrite, and 10 percent shale. A 4 ft to 6 ft thick medium bedded shale stratum occurs between approximately El. 495 and El. 505. Below El. 495, the bedded dolomite contains 5 to 10 percent gypsum, 40 percent anhydrite, and less than 5 percent shale.

In the station area, the core obtained from the borings indicated the bedded dolomite to be sound and unweathered. Typical rock core recovery ranged between 95 and 100 percent with the average being 98 percent.

c. Gypsum and anhydrite:

Generally, gypsum and anhydrite occurred in the dolomite as laminae with thicknesses generally less than 1/8 in. However, occasionally continuous strata of gypsum and anhydrite were penetrated. The maximum thickness of continuous gypsum strata was approximately 1 ft; the maximum thickness of continuous anhydrite strata was approximately 5 ft.

In the station area, core obtained from the borings indicated gypsum and anhydrite strata were sound and unweathered.

d. Shale:

The dolomite contains discontinuous, greenish black to black, medium hard, thin bedded shale strata above approximately El. 505. The thickness of these strata generally range from 0.5 ft to 1.5 ft.

Between El. 505 and El. 495, the borings indicated a 4 ft to 6 ft thick, continuous, dolomitic shale stratum. This stratum is greenish-black, mottled, medium hard to hard, and is medium bedded. There was essentially no indication of shale strata below El. 500. However, rock core obtained from the borings indicated the presence of thin (1/8 to 1/4 in. thick) shale partings of stringers below approximately El. 460. These shale partings are an indication that the borings penetrated into the Greenfield dolomite formation which underlies the Tymochtee formation.

In the station area, the core obtained from the borings indicated the shale strata were sound and unweathered.

e. Permeability:

The results of the in-situ pump-in bedrock permeability tests indicate the apparent permeability of the bedrock formation varies as follows:

Elevation	Range of Apparent Permeability
El. 560 to El. 535	$1x10^{-2}$ to $1x10^{-3}$ cm/sec
El. 535 to El. 510	$1x10^{-4}$ to $1x10^{-6}$ cm/sec
El. 510 to El. 490	$1x10^{-4}$ to $1x10^{-6}$ cm/sec
below El. 490	$1x10^{-4}$ to $1x10^{-6}$ cm/sec

The apparent coefficient of permeability of the bedrock above El. 535 measured in pump-out tests, using either a borehole and a series of piezometers or the test excavation and a series of piezometers, was found to be on the order of $1x10^{-2}$ cm/sec.

f. Evidence of solution activity:

1. Surface depression and Cooling Tower areas:

Evidence of solution activity was found in the surface depression area 400 ft south of the station area, and in a portion of the Cooling Tower area 1600 ft northwest of the station area. Geotechnical investigations were made in both areas. Subsequent to investigation of the Cooling Tower area, a remedial grouting program was implemented.

Results of these investigations indicated that solution activity occurs above El. 530. Solution activity generally occurs along joints and fissures in the bedded dolomite and shale strata immediately beneath the massive dolomite stratum. In both areas where solution activity was encountered, the bedded dolomite stratum, generally found above the massive dolomite stratum in the station area, was absent.

The size of the solution cavities ranged from relatively small fissures one inch wide and one to two feet long in the Cooling Tower area to large cavities up to 10 ft in diameter in the surface depression area. Small fissures were generally open. Large cavities were filled or partially filled with soil and bedrock material which either migrated or collapsed into the cavities. In the surface depression area, approximately 30 percent of the volume of all fissures and cavities was unfilled.

2. Station area:

Of the 20 borings and 86 rock probes made in the station area during preconstruction geotechnical investigations, 16 rock probes penetrated thin fissures in the bedrock. The width of the fissures ranged from 0.1 ft to 0.5 ft, except for two fissures which had widths of 0.9 ft and 1.2 ft. These fissures were not considered as significant solution activity.

A Bedrock Verification Program was implemented during foundation excavation to further explore the degree of bedrock solution activity in the station area. During the Bedrock Verification Program, both direct and indirect exploration methods were used. The detailed results of the Bedrock Verification Program are presented in Section 2C.5 of this Appendix and are summarized below.

Results of direct exploration including borings, probes, and geologic mapping indicated no significant solution activity in bedrock in the station area. Results of indirect exploration using geophysical methods including seismic, gravity, and resistivity surveys indicated no anomalous geophysical measurements indicative of significant solution activity in bedrock in the station area.

Consequently, based on (1) the absence of anomalous geophysical measurements, (2) the absence of significant solution activity in 71 borings and 198 rock probes made in the absence of the observation of significant solution activity on bedrock surfaces mapped during construction, it is concluded that the bedrock in the station area is free of significant solution activity.

g. Gas:

During the drilling of some borings and rock probes, hydrogen sulfide gas emitted from the bedrock portion of the boreholes. Hydrogen sulfide gas was also encountered during the test excavation and foundation excavation work. Chemical analyses of gas samples also indicated the presence of methane.

h. Bedrock socket-concrete bond strength:

Six field load tests were made to determine an allowable bedrock socket-concrete bond shear strength for use in the design of pier footings at the site. On the basis of these tests, in which maximum bedrock socket-concrete bond shear stresses of 40 k/ft^2 were applied, an allowable value of 36 k/ft^2 was selected to design the socket length for piers at the site.

2C.4.5 Dynamic Parameters of Till Deposit and Bedrock

1. In-Situ Investigation:

A seismic refraction survey was made to determine the in-situ compression-wave (P-wave) and shear-wave (S-wave) velocities of the soil deposits and bedrock. Dynamic parameters of these materials can be calculated from the measured values of P-wave and S-wave velocities.

A total of 26 shots were made for S-wave velocity measurements in conjunction with 26 shots made for P-wave velocity measurements. The locations of seismic survey lines are shown in Figures 2C.4-1 through 2C.4-3.

a. Till deposit:

The S-wave velocities of the till deposit could not be measured because the P-waves masked the S-waves. In the station area, the P-wave velocities of the till deposit are essentially uniform and average 5700 ft/sec. This value is considered representative for the till deposit.

b. Bedrock:

In the station area, the S-wave and P-wave velocities of the bedrock are also essentially uniform. In the station area, the average P-wave velocity of the bedrock is 12,700 ft/sec and the average S-wave velocity of the bedrock is 6700 ft/sec. These values are considered representative for the bedrock.

c. Dynamic parameters based on in-situ investigation:

The dynamic parameters (compression modulus, shear modulus, and Poisson's ratio) are calculated from the representative values of the P-wave and S-wave velocities and the representative values of the unit weight of the till deposit and bedrock. Because the S-wave velocity of the till deposit could not be measured, the value of the Poisson's ratio was assumed to be 0.4. The representative values of the unit weights were obtained in the laboratory; they are 136 lb/cu ft for the till deposit and 152 lb/cu ft for the bedrock. Representative values of the dynamic parameters are given as follows:

	Bedrock	Till Deposit
Measured Representative P-wave velocity V_p, ft/sec	12,700	5,700
Measured Representative S-wave velocity V_s, ft/sec	6,700	
Poisson's Ratio, μ	0.308 calculated	0.4 assumed
Calculated Compression Modulus E, 10^3 k/sq ft	550	64
Calculated Shear Modulus C, 10^3 k/sq ft	212	23

2. Laboratory Investigation:

Laboratory dynamic and static tests were made on representative samples of the till deposit and of the massive and bedded dolomitic bedrock. The dynamic tests included dynamic triaxial tests and ultrasonic dilatational wave velocity tests. Static tests included both triaxial and unconfined compression tests.

a. Till deposit:

Results of dynamic triaxial tests on samples of till deposit indicated that, as the axial strain increased during the test, the modulus value decreased and the damping ratio increased. As axial strains increased from $1x10^{-3}$ in/in to $4x10^{-3}$ in/in, the modulus decreased from $3x10^{3}$ k/sq ft to $1x10^{3}$ k/sq ft and the damping ratio increased from 0.10 to 0.17. These values are approximately equal to those considered representative for clay.

Results of ultrasonic velocity tests indicated an average ultrasonic wave velocity of 6200 ft/sec for the till deposit. A summary of the results of laboratory tests on samples of till deposit is shown in Table 2C.4-1.

b. Bedrock:

Tests were made on core samples of both the massive and bedded dolomite. Results obtained indicated that the modulus increases with increases in initial static confining stress, but decreases with increases in axial strain. This behavior is similar to that of a cohesionless soil (e.g., Hall et al 1963). Modulus values at low axial strain levels are essentially equal to values obtained from seismic measurements. Modulus values for the massive dolomite ranged from 1300 k/sq ft to 1800 k/sq ft. Modulus values for the bedded dolomite ranged from 300 k/sq ft to 1400 k/sq ft.

The values of Poisson's ratio for the massive dolomite are somewhat lower than those obtained from the seismic refraction survey. This is to be expected because values obtained from the seismic survey reflect the presence of fissures and other discontinuites in the bedrock and give higher values of Poisson's ratio. The values of Poisson's ratio of the bedded dolomite are very low and should not be used. Apparently, the bedding in the sample resulted in non-representative measurements of the radial strains at particular axial strain values and no reliable evaluation of Poisson's ratio could be obtained.

The damping values of the rock samples determined from the dynamic tests are approximately equal to those published for sand at corresponding strain values (Seed et al 1969).

The values of compression wave velocities determined in the ultrasonic velocity tests are slightly greater than those obtained from the seismic refraction measurements, again indicating the effect of fissures and discontinuities in the bedrock. The average ultrasonic wave velocity for massive dolomite was 18,200 ft/sec. The average ultrasonic wave velocity for bedded dolomite was 12,800 ft/sec.

A summary of the results of the laboratory tests on samples of massive and bedded dolomite is given in Table 2C.4-2.

3. Design Dynamic Parameters:

Based on the results of the in-situ investigation and the laboratory tests and considering the values of strains that the Maximum Probable (smaller) Earthquake and Maximum Possible (larger) Earthquake are expected to develop in the till deposit and bedrock, the following design values of dynamic parameters were selected.

a. Dynamic parameters applicable to the Maximum Probable Earthquake (smaller) earthquake:

	E, in 10^3 k/sq ft	G, in 10^3 k/sq ft	μ(1)	λ(2)
Till Deposit	34	12	0.4	0.04
Bedrock	470	180	0.3	0.01

b. Dynamic parameters applicable to the Maximum Possible Earthquake (larger) earthquake:

	E, in 10^3 k/sq ft	G, in 10^3 k/sq ft	μ(1)	λ(2)
Till Deposit	28	10	0.4	0.05
Bedrock	390	150	0.3	0.02

(1) μ = Poisson's ratio
(2) λ = Damping ratio

2C.4.6 Groundwater Conditions

1. Prior to Construction Dewatering:

Water level measurements made prior to the beginning of construction dewatering operations indicated that regional groundwater levels within the site area ranged from El. 571 to El. 572. Site area groundwater levels were approximately 1 ft to 2 ft above the mean Lake Erie water level. Regional groundwater gradients in the vicinity of the site area were 1 ft/mi to 2 ft/mi and were directed towards the lake. Bedrock elevation in the station area is approximately 10 ft below site groundwater levels. Because of the low permeability of the overlying glacial deposits, the more pervious bedrock aquifer is confined under a piezometric head of 10 ft.

2. During Construction Dewatering:

Operation of the construction dewatering system began on May 17, 1970 and continued until construction was completed to a point where dewatering could safely be stopped. The discharge rate remained essentially constant throughout this period at 350 gal/min. Groundwater levels in the station area were lowered to a minimum approximate El. 525 in the containment area. Results of intermittent piezometer water level measurements made subsequent to 17 May 1970 indicated that the maximum radius of influence of dewatering operations was approximately 5500 ft. Effects of construction dewatering operations on bedrock in the station area are discussed in Section 2C.5 of this Appendix.

No significant pumping of groundwater from the bedrock aquifer is expected during operation of the station. On completion of the excavation, water returned to normal levels within a year after dewatering operation was finished.

2C.4.7 Selected References

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TABLE 2C.4-1

Dynamic Parameters of Till Deposit Based on Laboratory Investigation

Sample Origin:

Three samples taken in borings 3-11 and 3-13 between El. 561 and El. 567 were tested.

Dynamic Triaxial Tests:

The samples were consolidated under an effective axial stress of approx 6 k/sq ft and effective lateral stress of approx 4 k/sq ft. For axial strains generally between $1x10^{-3}$ in/in and 2×10^{-3} in/in the deviator stresses generally varied between 0.7 k/sq ft and 2 k/sq ft. The calculated compression modulus E generally varied between 10^3 and 3×10^3 k/sq ft and the damping ratio between 0.10 and 0.17.

Ultrasonic Velocity Tests:

The representative average ultrasonic velocity is 6200 ft/sec.

Static Tests:

The compression modulus E calculated for the static triaxial tests and unconfined compression tests are approximately $E = 0.3 \times 10^3$ k/sq ft for the samples which had unconfined compressive strengths q_u= 4.2 t/sq ft and q_u= 6.4 t/sq ft. The compression modulus is approx 1.4×10^3 k/sq ft for the sample which had q_u = 8 t/sq ft.

TABLE 2C.4-2

Dynamic Parameters of Bedrock Based on Laboratory Investigation

	Massive Dolomite	Bedded Dolomite
Number of Samples Tested:	4	7
Sample Origin:		
Boring Number	2-1, 3-5, 3-10	2-1, 3-5, 3-13
Elevation Range	551-555	498-556
Dynamic Triaxial Tests:		
Approx Range Consolidation Stresses:		
Axial σ1c, k/sq ft	6-50	6-60
Lateral σ3c, k/sq ft	6-7	6-14
Approx Range Deviator Stresses, k/sq ft	1.5-30	1.5-30
Axial Strains, in/in	$2x10^{-6}$ to $10x10^{-6}$	$5x10^{-6}$ to $20x10^{-6}$
Radial Strains, in/in	10^{-6} to $4x10^{-6}$	$2x10^{-6}$ to $4x10^{-6}$
Compression Modulus E, in 10^3 k/sq ft	1300-1800	399-1400
Shear Modulus G, in 10^3 k/sq ft	550-690	100-400
Poissons Ratio, μ	0.20-0.28	0.12-0.20*
Damping Ratio, λ	0-0.02	0-0.02
Ultrasonic Velocity Tests:		
Representative Average Longitudinal Velocity V_L ft/sec	18 200	12 800
Static Tests:		
Compression Modulus from Triaxial Test E, in 10^3 k/sq ft	1500	500
Compression Modulus from Unconfined Compression Test E, in 10^3 k/sq ft	770	350
Unconfined Compressive Strength q_u, t/sq ft	1500	750

* not reliable

2C.5.0 Results of Bedrock Verification, Groundwater, Monitoring, and Partial Class I Earthwork Quality Assurance Programs

2C.5.1 Introduction

A Geotechnical Quality Assurance Program (GQAP) was prepared and then implemented during the construction of Davis-Besse Nuclear Power Station Unit No.1. The GQAP consisted of three parts: (1) a Bedrock Verification and Remedial Treatment Program; (2) a Groundwater Monitoring Program; and (3) a Class I Earthwork[1] Quality Assurance Program.

A Geotechnical Quality Assurance Manual (GQAM), dated 2 October 1970, was prepared by Woodward-Moorhouse & Associates, Inc. (WMAI, formerly Woodward Clyde & Associates, Inc.) in accordance with the general requirements of AEC Regulation 10CFR50. The GQAM presented: (1) the purpose and objectives of the GQAP; (2) the particular requirements of each of the three parts of the GQAP; (3) the quality control and quality assurance procedures to be followed during Implementation of each part of the GQAP: and (4) the pertinent data forms to be used during implementation of each part of the GQAP.

The GQAM was reviewed and orally approved by members of the AEC Division of Compliance during November 1970 and the Toledo Edison Company (TED), Quality Assurance Engineer in December 1970.

This section presents: (1) a summary of the requirements presented in the GQAM for each part of the GQAP; (2) an analysis of the results obtained during implementation of the GQAP; and (3) conclusions concerning (a) the presence of significant solution activity in the bedrock in the station area, (b) the effect of construction dewatering operations on the bedrock in the station area, and (c) the quality of the class I earthwork construction.

2C.5.2 Bedrock Verification Program

1. General:

Prior to construction, subsurface investigations were made at the Davis-Besse Nuclear Power Station site. The locations of preconstruction NX-rock core borings and rock probes made in the station area during those investigations are presented in Figure 2C.5-1.

Neither significant fissures nor cavities were encountered in the preconstruction investigation of the station area. However, because of the presence of cavities in the surface depression area approximately 400 ft south of the station area, it was considered possible that significant fissures 1-ft to 2-ft wide and cavities several cubic yards in volume could exist in the station area.

For this reason, it was considered necessary that the degree of the bedrock solution activity in the station area be further explored by a Bedrock Verification Program made during foundation excavation.

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[1] Class I earthwork discussed in this section pertains only to Class I earthwork in the Intake Forebay Dike area.

The detailed requirements of the Bedrock Verification Program were presented in the GQAM. The Bedrock Verification Program, as presented in Section 3 of the GQAM, consisted of two parts. Part A was an exploration program consisting of the following four sub-parts: (1) detailed inspection and mapping of the bedrock as it was exposed in the excavation; (2) drilling of NX-rock core borings; (3) drilling of rock probes in areas of joint concentrations and below the bottom of drilled pier foundations; and (4) making geophysical surveys to measure compression-wave velocity, the local gravitational field, and the resistivity of the bedrock. The purpose of subparts 1, 2, and 3 was to locate, by direct methods, areas of solution activity. The purpose of sub-part 4 was to locate areas of anomalous geophysical measurements which would then be considered areas of suspected significant solution activity. Any area of solution activity or suspected solution activity would then be investigated further during part B of the verification program by a detailed local exploration program in order to determine the vertical and horizontal extent of areas of significant solution activity. A flow chart presenting the outline of the Bedrock Verification Program is shown in Figure 2C.5-2. The following text presents: (1) the detailed requirements of the Bedrock Verification Program; (2) an analysis of results obtained from the Bedrock Verification Program; and (3) conclusions based on these results.

2. Summary of the Requirements of the Bedrock Verification Program:

a. Direct methods:

1) Boring program:

A boring program consisting of a minimum of 49 NX-rock core borings was proposed. The borings were to be logged and the cores inspected and classified. The purpose of the borings was to determine the vertical extent of any significant bedrock solution activity. Forty-six of the borings were to extend to El. 470 and three borings were to extend to El. 390.

The borings were to consist of both inclined and vertical borings. The locations of inclined boring were to be selected by considering the results of the geologic mapping program.

2) Geologic mapping program:

As the bedrock was exposed in the excavation, it was to be washed by high pressure water jet, inspected in detail, photographed, and mapped. The mapping was to be done at a scale of 1 in. = 20 ft and the mapping notes were to include width and condition of joints and fissures, filling material, and other pertinent information indicative of solution activity. As the bedrock was excavated, the exposed rock surfaces were to be mapped and transparent overlays were to be prepared. The data were to be analyzed to determine the orientation of the major joint sets and the location of joint intersections. This information was to be used as an aid in designing or modifying the boring program, rock probe program, and geophysical surveys.

3) Rock probe program:

The rock probe program was to consist of rock probes drilled from foundation grade in areas where bedrock surface mapping indicated the possibility of solution activity or structural conditions favoring solution activity (ie, joint concentrations). In addition, as a final step in part A of the Bedrock Verification Program, rock probes were to be drilled beneath the isolated pier footings; these probe holes were then to be grouted (see Figure 2C.5-2).

Areas of bedrock at foundation grade in which two or more major joint intersections occurred within horizontal distances of ten feet were to be explored with one or more rock probes. The probe would be drilled to a depth of 20 ft below foundation grade using an air-driven percussion drill. Drilling inspection of the probe holes was to include observation of the penetration rate of the drill rod, the behavior of the drill rod, and the character of the rock cuttings.

If no direct or indirect evidence of significant solution activity was found during part A, no additonal verification work was proposed under the mat foundations (see Figure 2C.5-2). In the case of isolated footing foundations, however, (ie, pier footing foundations), a rock probe was to be drilled to a depth of twice the design diameter beneath the design bottom elevation of the pier footing. The drilling was to be inspected by an Inspector, Field Engineer, or Field Geologist. After drilling, an Inspector was to supervise filling the probe hole with cement-water grout. The basic grout was to consist of cement and water mixed at a 1:1 ratio by volume. The grouting pressure was to be 1 lb/sq in. per foot of depth below the design bottom of the pier, and the pressure was to be applied until the volume of grout pumped per unit of time into the hole was less than 1 cu ft/min for a period of ten minutes.

b. Indirect methods:

1) Seismic velocity survey:

Compression-wave velocities of the bedrock were to be determined between selected NX-drill holes and 3-in.-diameter percussion drill holes by means of a series of cross-hole seismic velocity measurements.

Seismic velocity measurements were to be made at three elevations between the shot points and recording stations: El. 490, El. 520, and at an elevation 5 ft below foundation grade. The coverage provided by this seismic survey would result in a minimum of 87 compression-wave velocity determinations.

The computed compression-wave velocity data were to be plotted on three 1 in. = 50 ft plans of the station area. Velocity data were to be examined and a decision made to define anomalous velocity values. The judgement decision to select anomalous velocity values was to be based on both previous and additional compression-wave velocity measurements made in the surface depression area south of the station area where solution activity was previously found. In addition, the results of preconstruction field and laboratory seismic bedrock investigations made in the site area were also to be used to assist in evaluating the seismic velocity measurements.

2) Gravity survey:

Measurements of the earth gravitational field were to be made in the station area using a Lacoste-Romberg Gravimeter. The foundation excavation for the Containment area was to be surveyed by a series of traverses oriented in the northeast and northwest directions, which are the directions of the predominant joints. Spacing between traverses was to be 15 ft. Gravity measurements were to be made every 15 feet along a traverse. In the mat area of the Auxiliary Building, measurements were to be made at major wall intersections, at the wall midpoints between major wall intersections, and in the center of major floor bays. In the footing area of the Auxiliary Building and in the Turbine Building areas, measurements were to be made under each isolated footing. In the Intake Structure area, the traverses were to be at a spacing of 20 ft with measurements every 20 ft along a traverse.

Gravity measurements were to b compiled and corrections made, as required, to obtain Bouguer gravity values. Corrected data were to be plotted on a 1 in. = 20 ft plan and an isogal contour map prepared. This map was examined and a judgement decision made regarding areas in which anomalous gravitational field variations occurred. The judgement decision regarding areas of anomalous values was aided by a series of calibration measurements made in the surface depression area. The expected significant gravitational difference which would indicate an anomaly was estimated to be on the order of 0.1 milligals.

3) Resistivity survey:

Fixed-depth resistivity measurements were to be made in the station area with a Megger Earth Tester using the Wenner electrode configuration. The Wenner configuration consists of four electrodes located on a straight line. The distance between all electrodes is equal and referred to as the electrode spacing. Electrode spacings were to be 20 ft and 40 ft.

The foundation excavation of the Containment Structure was to be surveyed by a series of traverses oriented in the northeast and northwest directions, which are the directions of the predominant joints. Spacing between traverses was to be 15 ft. Resistivity measurements were to be made every 15 ft along a traverse. In the mat area of the Auxiliary Building, measurements were to be made at major wall intersections, at the wall midpoints between major wall intersections, and in the center of major floor bays. In the footing area of the Auxiliary Building and in the Turbine Building area, measurements were to be made under each isolated footing. In the Intake Structure area, the traverses were to be at a spacing of 20 ft with measurements every 20 ft along a traverse.

The resistivity measurements were to be plotted on a plan drawn to a scale of 1 in. = 20 ft and contours of equal resistivity were to be inferred. These contours were to be examined and a judgement decision made to locate areas with anomalous resistivity values. The judgement decision was to be based in part on the results of a series of resistivity traverses which were to be made in the surface depression area south of the station area.

It was expected that: (1) in areas more than 20 ft from (a) the vertical rock faces of the Containment and Auxiliary Building excavations, or (b) the dewatering system header pipe, that a significant anomalous resistivity value would be on the order of 5,000 ohm-cm above or below that of the average resistivity measured in a particular foundation area; and (2) in areas less than 20 ft from excavation faces or dewatering header pipes, the level of scatter in the measured apparent resistivity would increase to such a degree that the variation in measured apparent resistivity resulting from the presence of solution activity would probably be masked.

3. Presentation and Analysis of Results Obtained During the Bedrock Verification Program:

a. Direct methods:

1) Boring program:

A total of 51 NX-rock core borings were made in the station area. The location and orientation of the 51 borings are presented in Figure 2C.5-3. A summary of the boring program by general boring location is as follows:

Location	Total No. of Borings	No. of Borings and Approximate Termination El. — Vertical	Inclined
Containment Area	11	6 to El. 470 1 to El. 390	3 to El. 470 1 to El. 390
Auxiliary Building (Mat Area)	11	6 to El. 470	4 to El. 470 1 to El. 390
Auxiliary Building (Footing Area)	8	2 to El. 470	6 to El. 470
Turbine Area	11	8 to El. 470	3 to El. 470
Intake Structure Area	6	0	5 to El. 470 1 to El. 390
Office Building Area	4	1 to El. 470	3 to El. 470
TOTAL	51		

Table 2C.5-1 presents asunimary of the pertinent data obtained from each of the 51 borings.

The borings were inspected by a Field Inspector. The Inspector recorded: (1) drill time for each 0.1 ft of drill penetration; (2) length of rock cores recovered; and (3) comments concerning the color of drill water, presence of voids, and other information not shown directly by recovered cores, but indicative of subsurface conditions. The rock cores were placed in core boxes by the Inspector; the core boxes identified; and the rock cores were later geologically classified by Field Geologists.

Data obtained during the boring inspection were recorded on a field boring log. Data obtained from geologic classification of the rock cores were recorded on a rock classification data sheet (RCDS). The field boring logs and RCDS are presently on file at the TED on-site office. Design boring logs were prepared for each of the 51 borings and are also on file at the TED on-site office. Design boring logs and RCDS for borings B7-30 and B7-51 (which are typical inclined and vertical borings, respectively) are presented in Figures 2C.5-4 through 2C.5-20.

Analysis of the results obtained from the 51 borings made indicates that: (1) the percent core recovery ranged from 45 to 100 percent with the average being 98 percent; and (2) there were no zones of low drill times or sudden drops of the drill steel encountered in the borings (low drill times are those less than 2 sec per 0.1 ft); and (3) no evidence of significant solution activity was observed in the rock cores recovered.

2) Geologic mapping program:

Geologic maps at a scale of 1 in. = 20 ft were prepared of: (1) the bedrock surface exposed after excavation of the soil deposits; and (2) the bedrock surfaces exposed after rock excavation. In addition, geologic maps at a scale of 1 in = 5 ft were prepared of the pre-split rock excavation walls in the station area. After the bedrock surfaces and pre-split walls were exposed, they were washed where necessary using a high-pressure water jet. The exposed surfaces were examined and mapped by a Field Geologist.

The following data were obtained during geologic mapping of the bedrock surfaces: (1) location, size, orientation, and extent of surface joints; (2) geologic classification and plan extent of bedrock deposits existing on the mapped surface; and (3) geologic classification of joint filling materials.

The following data were obtained during geologic mapping of the pre-split excavation walls: (1) location, size, orientation, and extent of joints; (2) elevation of the interfaces between the main bedrock strata exposed on the walls; and (3) geologic description of the main bedrock strata.

Reduced-scale maps are presented in Figures 2C.5-21 through 2C.5-25.

In addition to field mapping, detailed photographs were taken of the bedrock surfaces and pre-split walls. Figure 2C.5-26 presents an example of the bedrock surface photographs taken during the mapping program.

Analysis of data obtained from the geologic mapping indicates: (1) the major primary joint set has a strike of approximately N45°E and the secondary joint sets have strikes of N50°W and N90°W; (2) joints mapped were typically vertical; (3) approximately 10 percent of the joints mapped on the bedrock surface (El. 560±) (see Figure 2C.5-21) were open and had indications of minor solution activity. This minor solution activity was confined to the upper three feet of bedrock (see (5) below); (4) approximately 90 percent of the joints on the bedrock surface (El. 560±) (see Figure 2C.5-21) were typically less than 0.1 ft wide, filled with till or satin spar gypsum, and had no indications of solution activity; (5) joints mapped on excavated bedrock surfaces and pre-split excavation walls were typically less than 0.05 ft wide, were filled with satin spar gypsum, and had no indications of solution activity below a depth of 3 ft below the top of the bedrock surface (See Figs. V-9, V-10, and V-11); and (6) there were no indications of significant continuity of joints with depth when comparing the location of joints mapped at the bedrock surface (El. 560) with those mapped on excavated bedrock surfaces (El. 542 to El. 528), (see Figures 2C.5-21 and 2C.5-22).

3) Rock probe program:

A total of 114 rock probes were made; two were made at foundation grades where the geologic mapping of bedrock surfaces indicated a significant concentration of joint intersections (see Figure 2C.5-3), and the remaining 112 were made beneath pier footings to be constructed in the Turbine, Office, and Auxiliary Building areas. Probes made at pier locations were pressure grouted with a cement-water grout mixture; probes made at foundation grade were backfilled with a cement-water grout.

The two rock probes made at foundation grade were made to a depth of 20 ft beneath the planned foundation grade at an angle of 20 degrees to the vertical and oriented in a direction approximately normal to the strike of the joints. The rock probes which were made at pier

locations were drilled vertically beneath the design bottom elevation of the pier to a minimum depth of twice the design diameter.

Inspection of drilling and grouting operations was provided by a Field Inspector. Data obtained during inspection were as follows: (1) drill rate for each 0.5 ft of drill penetration; (2) sudden drops of the drill steel; and (3) other indications of the presence of significant solution activity.

For the rock probes made at pier locations, the data obtained during grouting operations were as follows: grouting pressure, packer depth, grout mix design, and grout take rate.

Figure 2C.5-13 presents plots shoving NX-rock core drill times obtained in rock not containing significant solution activity and air-track drill times obtained in the rock probe adjacent to this NX-rock core boring. Analysis of the data presented in this figure indicates that air-track drill times range from 0.05 to 0.25 times the NX-rock core drill times obtained at equal elevations.

Minimum NX-rock core drill times of 40 sec per 0.5 ft were obtained, and on the basis of the comparison shown in Figure 2C.5-27, it was concluded that air-track drill times of 2 sec per 0.5 ft or greater indicated that the rock penetrated did not contain significant solution activity. In additon, the subsequent grout takes were evaluated in conjunction with the drill times to assess the possibility that solution activity had been encountered in the probes.

The data obtained during inspection of the rock probes were recorded on rock probe drilling and grouting logs; the rock probe drilling and grouting logs are on file at the TED on-site office. Pertinent rock probe, drilling, and grouting data have been summarized and are presented in Tables 2C.5-2 through 2C.5-4.

Analysis of the data obtained during drilling and grouting of the rock probes indicates that: (1) during drilling of the probes, there were no sudden drops of drill steel indicative of voids nor were there any zones on rock with drill times less than 2 sec/0.5 ft; and (2) there were no excessive grout takes indicative of possible solution activity (see discussion below).

Grout takes in excess of 10 cu ft were noted at nine pier locations. All of these locations were close to either the dewatering system header pipe or the vertical face of a nearby rock excavation. Evidence such as discoloration of dewatering system discharge water and the leakage of grout through small horizontal fissures on the face of rock excavations was noted during grouting. No low drill times were noted in these nine rock probes. Consequently, it is concluded that the higher than typical grout takes encountered in these nine probe holes were the result of a flow of grout into the dewatering system or a loss of grout through open faces of excavated bedrock surfaces, and are not indicative of significant solution activity.

b. Indirect methods:

1) General:

In addition to the direct methods of bedrock exploration described previously, the bedrock in the station area was explored by three geophysical methods: seismic velocity, gravity, and resistivity. The purpose of using the indirect methods was to locate areas of suspected significant solution activity the bedrock below the station area. Suspected areas would be defined by anomalous geophysical measurements and, if found, would be extensively investigated using direct methods of exploration such as closely spaced borings and/or local excavations.

The criterion for selection of an anomalous geophysical measurement for each method was based on the following general considerations:

a. The criterion must be conservative because it is used to decide the need for additional subsurface investigation.

b. The criterion can best be established by examining the entire data population.

c. The criterion must consider the accuracy of the measurement being made; including systematic errors resulting from instrument and technique capability.

d. The criterion must consider the effect on the measurement of conditions not related to solution activity (i.e., the effects of groundwater, topography, and bedrock structure).

The following text presents the criterion for selection of anomalous geophysical measurements for each geophysical method and the analysis of results obtained.

2) Seismic velocity survey:

(a) Criterion for selection of anomalous compression-wave velocity values:

The purpose of the cross-hole seismic survey was to detect areas of bedrock having low compression-wave velocities (henceforth, compression-wave velocity is referred to as seismic velocity). A low value of seismic velocity could indicate that a void or less dense rock mass (caused by significant solution activity) existed between the shot point and recording station locations.

To aid in the judgement decision regarding anomalous seismic velocity values, a broadside seismic calibration survey was made in an area of known solution activity (the surface depression area located 400 ft south of the station area). The calibration survey was made prior to making the cross-hole seismic survey in the station area. The results obtained are presented in Figure 2C.5-28. Analysis of the results indicates that seismic velocities computed from arrival times recorded in the center six geophones (P4 through P9) were typically about 9000 ft/sec. This velocity is consistently lower than the velocities (10,000 to 11,000 ft/sec) computed from arrival times recorded in the outside geophones (P1 through P3 and P10 through P12).

Seismic velocities determined in rock free of significant solution activity during the preconstruction field and laboratory bedrock investigations were in all cases greater than 10,000 ft/sec.

Based on analysis of the data obtained from the calibration survey and taking into account the general considerations for selection of criterion for determination of anomalous values, it was concluded that a seismic velocity of less than 9000 ft/sec would be an anomalous value.

(b) Procedure and analysis:

Ninety-one cross-hole seismic velocity measurements were made using six shot point locations and from one to nine recording stations. The shot elevations were at El. 490, El. 520, El. 535, and El. 550. The locations of the shot points and recording stations, the directions and elevations of the measurements, and the resulting seismic velocity measurements are presented in Figures 2C.5-29 through 2C.5-32.

For each of the cross-hole seismic velocity measurements, the elapsed time between shot detonation and compression-wave arrival at the geophone was recorded on photographic paper. After the shot was made, pertinent identification data were marked on the photographic shot record by the Geologist making the measurements.

The cross-hole seismic velocity was calculated by dividing the distance between the shot point and recording station by the elapsed time between shot detonation and the first compression-wave arrival. This calculated seismic velocity was plotted as a velocity vector on a 1 in. = 50 ft location plan prepared for the particular shot elevation (see Figures 2C.5-29 through 2C.5- 32).

Analysis of the data obtained from the cross-hole seismic survey indicates that: (1) the calculated seismic velocities ranged from 10,180 to 20,440 ft/sec; (2) for distances between shot point and recording station less than 150 ft, the calculated seismic velocities were generally between 10,000 and 13,000 ft/sec; and for distances between shot points and recording station greater than 150 ft, the calculated seismic velocities were generally between 13,000 and 20,000 ft/sec; (3) the apparent increase in calculated seismic velocity at the greater recording point distances results from refraction of the compression wave through a continuous, medium bedded shale bed (having a high seismic velocity) located between El. 500 and El. 510 (see Figures 2C.4-6, 2C.4-7, and 2C.4-8); and (4) no anomalous seismic velocities were obtained (see Figures 2C.5-29 through 2C.5-32).

3) Gravity survey:

(a) Criterion for selection of anomalous gravity values:

The principle of the gravity method is the measurement of the force of attraction between two masses. The force varies proportionately to the magnitude of' the masses and inversely to the square of the distance between them. In the gravity survey, the gravimeter mass is kept constant. Variations in the attractive force measured by the gravimeter result from changes in mass distribution or density of soil and rock in the immediate vicinity of the instrument.

For purposes of the Bedrock Verification Program, the gravity method is used to detect areas of low relative Bouguer gravity, if such areas exist. Such low values would be indicative of mass deficiencies in the underlying bedrock which could be a result of significant solution activity.

Prior to implementation of the GQAP, it was estimated that a gravitational difference of 0.1 milligal would be a conservative criterion for selection of anomalous gravity values. However, on the basis of the analysis of data obtained from the calibration surveys in the surface depression area and station area, and the gravity measurements in the station area, it was concluded that, for this site, this criterion is unconservative. The following paragraphs present the results of the calibration surveys and the revised, more conservative, criterion for selection of anomalous gravity values.

Two calibration surveys were made (1) ever a 4-ft-diameter pier excavation in the station area; and (2) over an area of known solution activity in the surface depression area. The results of the pier excavation calibration survey are presented in Figure 2C.5-33. The results of the surface depression area calibration survey are presented in Figure 2C.5-28.

Analysis of the results of the calibration surveys indicates that: (1) calibration survey measurements made over the 4-ft-diameter pier excavation indicated a 0.055 milligal. Bouguer anomaly; theoretical analysis results in a calculated anomaly of 0.050 milligal; (2) the calibration

survey measurements made in the surface depression area indicated a 0.085 milligal Bouguer anomaly at approximate coordinate location N9500 E102050. An air-track drill probe was made at coordinate location N9484 E10252 (approximately 16 ft from the location of the maximum anomaly). The probe penetrated a 4-ft void between El. 556 and El. 552 (these elevations correspond to depths of 19 ft and 23 ft below the elevation at which the gravity survey measurements were made).

On the basis of the results of the calibration surveys, it was concluded that: (1) the instrument and technique used to make the gravity measurements indicated anomalies in area where mass deficiencies were known to occur; and (2) the original estimate of 9.100 milligals as a criterion for selection of anomalous gravity values was too large.

Based on analysis of the results of the calibration surveys, the gravity measurements made in the station area, and the errors associated with the gravity survey (i.e., reading errors, leveling errors, and errors in the topographic corrections), and taking into account the general considerations for selection of criterion for determination of anomalous values, it was concluded that for topographically corrected Bouguer gravity readings, a gravity gradient greater that 0.0015 milligals/ft between adjacent stations (typically 15 ft to 20 ft) would be an anomalous value.

(b) Procedure and analysis:

A total of 319 gravity measurements were made at the plan foundation grade in the station area. Measurements were made using a LaCoste-Romberg Model G gravimeter.

The data obtained during gravity measurements made in the station area consisted of: (1) the coordinate location and elevation of the gravity station; (2) the time of day at which the gravity reading was made; and (3) a gravimeter reading. The gravimeter reading was reduced to a computed value of relative Bouguer gravity by normalization to a given elevation, latitude, and time datum by the application of standard slab, free-air, and drift corrections. Topographic corrections were not initially included. For a discussion of these standard gravity survey corrections, refer to Parasnis (1962). The field gravity data and the calculated values of normalized relative Bouguer gravity (without topographic corrections) were recorded on gravity data reduction sheets. The relative Bouguer gravity values were plotted on a plan and iso-gal contours were drawn to allow immediate field evaluation of the gravity survey results prior to application of topographic corrections. The resulting iso-gal contour map is presented in Figure 2C.5-34.

Iso-gal contours prepared using relative Bouguer gravity readings without topographic correction should follow the effects of topographic conditions with gravity readings decreasing as the reading stations approach topographic irregularities such as the pre-split excavation walls and soil slopes. Analysis of the iso-gal contours on Figure 2C.5-34 indicates that: (1) the gravity contours closely follow the effects of the topography in the station area (see Figure 2C.5-35 for a topographic map of the station area excavation prepared at the time of the gravity survey); and (2) there are no irregular low values of relative Bouguer gravity which would indicate the presence of significant solution activity.

After completion of the field gravity measurements, Tidelands Geophysical Inc. of Houston, Texas, computed the topographic corrections for the values of relative Bouguer gravity. The resulting iso-gal contour map is presented in Figure 2C.5-36. Analysis of these iso-gal contours and the topographically corrected Bouguer gravity tends to increase in a northwest direction across the station area; (2) there are no isolated low values of topographically corrected

Bouguer gravity; (3) the maximum difference in the topographically corrected Bouguer gravity across the station area (approximately 600 ft) is only 0.060 milligals; (4) the gravity gradients between adjacent readings range from 0.0001 to 0.001 milligals/ft; and (5) there are no areas in which the gravity gradient between adjacent readings is greater than 0.0015 milligals/ft.

It is concluded from this analysis that: (1) because of the uniformity of the iso-gal contours and the small difference in gravity values across the station area, the contours represent regional gravity gradients; and, (2) the results of the gravity survey in the station area, indicate the presence of uniform bedrock free of significant solution activity.

4) Resistivity survey:

(a) Criterion for selection of anomalous resistivity values:

Apparent resistivity values, obtained using the Wenner electrode configuration, represent the average resistivity of a semi-elliptical volume of rock located between the center pair of four equally spaced electrodes. The effective depth of the measurement is approximately equal to the electrode spacing. The purpose of the resistivity survey is to detect areas of bedrock having significantly higher or lower than average resistivity values. Water-filled voids in the bedrock will be areas of lower than average resistivity, while air-filled voids will be areas of higher than average resistivity.

Resistivity measurements made in the station area were influenced by the following factors that are not related to solution activity: (1) variations in the level of the groundwater with respect to the elevation at which the measurement was made; (2) groundwater flow to and through the dewatering system; (3) the average porosity of the bedrock; and (4) the proximity of the reading station to topographic features such as vertical faces of rock excavations or soil slopes. A discussion of each of these factors is presented in the following paragraphs.

The apparent resistivity is strongly affected by the presence or absence of water. If the voids (i.e., pores, joints, fissures, or solution cavities) in the bedrock are air filled, the measured resistivity will be greater than that for the intact bedrock because of the high resistivity of air. If the voids in the bedrock are water filled, the measured resistivity will be less than that for the intact bedrock because of the relatively low resistivity of water. Consequently, the measured resistivity of a bedrock formation for a given electrode spacing decreases as the level of groundwater rises in the vicinity of the measurement station (Meidav et al 1960 and Cook et al 1954).

The apparent resistivity is affected slightly by flowing groundwater. Flowing groundwater causes the movement of ions in the water which results in the generation of a direct current. If resistivity measurements are made near a zone of flowing groundwater, the resulting direct current may cause a decrease or increase in the measured apparent resistance value.

The apparent resistivity of rock is affected by the porosity of the rock. If the rock mass is saturated, the resistivity decreases with increased porosity because of the presence of the water; if the rock mass is air filled, the resistivity increases with increased porosity because of the presence of air.

The apparent resistivity is also affected by topographic features in the vicinity of the reading station. The theory of apparent resistivity assumes the measurements are made on the surface of a semi-infinite half space. Topographic features, such as excavations or cut slopes near the reading station, represent variations from this assumption. Cut slopes above the elevation of

the measurement station tend to decrease the apparent resistivity because there are materials (i.e., either rock or soil) above the elevation of the reading station with a finite resistivity. Open excavation beneath the elevation of the measurement station tend to increase the apparent resistivity because there is material (i.e., air) beneath the elevation of the reading station with an essentially infinite resistivity.

A calibration resistivity traverse was made across an area of known solution activity in the surface depression area to aid in the judgement decisions regarding anomalous resistivity readings. The electrode spacings used were 25 ft and 50 ft because the rock in this area was covered by approximately 13 ft of soil deposits. The data obtained from this survey is presented in Figure 2C.5-28. Analysis of these data indicates that the resistivity values obtained for both the 25-ft and 40-ft electrode spacings, at sta 2 + 25 along the traverse (see Figure 2C.5-28), were from 1000 to 2000 ohm-cm below the average values for the balance of the traverse. Boring B2-8, previously made near sta 2 + 25 along the traverse line, indicated a 4-ft zone of solution activity between El. 550 and El. 546. The zone of solution activity was saturated at the time of the measurement.

Using the two-layer resistivity curve (Roman 1934) and based upon an assumed resistivity of 1500 ohm-cm and a thickness of 13 ft for the soil deposits along the calibration traverse, the resistivity of the underlying bedrock was calculated. The results of the calculations indicate that for the assumed conditions, the bedrock resistivity ranged from 22,000 to 35,000 ohm-cm with an average value of 28,500 ohm-cm. The calculations also indicate that the measured 1000 to 2000 ohm-cm decrease in apparent resistivity would result from a 6000 to 8000 ohm-cm decrease in the resistivity of the underlying bedrock.

On the basis of results obtained from the calibration survey, it was concluded that the resistivity equipment and technique were suitable for detection of solution activity in the bedrock at the site.

Prior to implementation of the GQAP (and as presented in the GQAM) it was expected, based on the anticipated general effects of groundwater and topographic conditions in the station area on the resistivity measurements, that: (1) in areas more than 20 ft from topographic anomalies such as rock excavations and soil slopes or the dewatering header, that a significant anomalous resistivity value would be on the order of 5000 ohm-cm above or below that of the average resistivity measured in a particular foundation area; and (2) in areas less than 20 ft from these features, the level of scatter in the measured apparent resitivity would increase to such a degree that the variation caused by solution activity would be masked.

Based on analysis of: (1) the results obtained from the calibration survey made in the surface depression area: (2) the results obtained from resistivity measurements made in the station area; and (3) the mechanical errors associated with the resistivity survey (i.e., imperfect coupling of electrodes with the bedrock and reading errors); and (4) taking into account the general considerations for selection of criterion for the determination of anomalous values, it was concluded that the criterion presented in the GQAM is suitable for selection of anomalous resistivity values if the ±5000 ohm-cm variation is applied to the average resistivity gradient rather than the numerical average resistivity value for particular foundation areas.

(b) Procedure and analysis:

Resistivity measurements were made at 275 locations at foundation grade in the station area. The resistivity measurements were made using a Megger Earth Tester and the Wenner electrode configuration.

The data obtained during the resistivity survey in the station area consisted of: (1) a point designation number; and (2) a measured resistance between the center electrodes of the Wenner configuration. Field data were recorded on resistivity data sheets. The value of resistivity was calculated using the data obtained and was also recorded on resistivity data sheets.

Equi-resistivity contour maps have been prepared from the data obtained from the 20-ft and 40-ft fixed-depth resistivity measurements made in the station area. These contour maps are presented at reduced scale in Figures 2C.5-37 and 2C.5-38.

In addition, after reviewing the equi-resistivity contour maps, two selected sections were prepared to assist in evaluating the results of the resistivity survey. These sections present: (1) topographic, groundwater, and geologic conditions along the section lines; and (2) the corresponding 20-ft and 40-ft fixed-depth resistivity measurements. These sections are presented in Figure 2C.5-39.

Typical values of resistivity determined from analysis of the equi-resistivity contours presented in Figures 2C.5-37 and 2C.5-38 and the pertinent topographic and groundwater conditions for the various building areas are summarized as follows:

			Typical Apparent Resistivity Readings	
Area	Approximate Surface Elevation, ft	Approximate Water Level Elevation, ft	20-ft Electrode Spacing ohm-cm	40-ft Electrode Spacing ohm-cm
Turbine and Office Buildings	560	542	45,000	35,000
Auxiliary Building (Footing Portion)	559	545	40,000	30,000
Auxiliary Building (Mat Portion)	542	542	17,500	17,500
Containment	540 to 528	540 to 528	15,000	15,000
Intake Structure	543	543	35,000	22,000

The sections presented in Figure 2C.5-39 present examples of the variation in resistivity measurements resulting from geologic, groundwater, and topographic conditions existing in the various building areas at the time the resistivity measurements were made.

Review of the data presented in Figure 2C.5-39 indicates, as would be expected by theoretical analysis, the resistivity readings obtained with 40-ft electrode spacing are, in general, less affected by groundwater and topographic conditions than those readings obtained with 20-ft electrode spacing.

Data presented in Section 1-1 of Figure 2C.5-39 indicates that: (1) resistivity values decreased in rock excavation areas (i.e., the containment and Auxiliary Building areas) where the groundwater level was close to the elevation where the measurements were made; (2)

resistivity values for the 40- ft electrode spacing were erratic in the Containment Building area near the lower stage dewatering system header and discharge pipes; and (3) resistivity values were within 5000 ohm-cm of an average resistivity gradient line at points more than 20 ft from the limits of the dewatering system and air- or water-filled excavations.

Data presented in Section 2-2 of Figure 2C.5-39 indicates that: (1) resistivity values increased where resistivity measurement stations were adjacent to air-filled dewatering system trench excavation (i.e., in the southern part of the Turbine Building area); (2) resistivity values for the 20-ft electrode spacing decreased where resistivity measurement stations were adjacent to the water-filled dewatering trench excavation (i.e., in the northern portion of the Turbine Building area); (3) the plan limits of the zone of resistivity readings affected by the water-filled dewatering trench were larger in the northeastern corner of the Turbine Building area than in the northwestern corner of the building area because, at this point, the northern dewatering trench excavation intersected the eastern dewatering trench excavation (both water filled), and the size of the zone of bedrock wetted by seepage from the dewatering trench to the groundwater level increased; and (4) resistivity values are well within 5000 ohm-cm of an average gradient line at points more than 20 ft from the air-filled dewatering trench excavation in the southern portion of the Turbine Building area and 40 ft from the water-filled dewatering trench excavations in the northeastern corner of the Turbine Building area.

Analysis of the equi-resistivity contour maps (see Figures 2C.5-37 and 2C.5-38) indicates that outside the dimensional limits previously given, resistivity values are within ±5000 ohm-cm of the average resistivity gradient for each foundation area.

It is noted that the typical resistivity values obtained in the footing portion of the Auxiliary Building area were 10,000 ohm-cm lower than those obtained in the Turbine Building area. Resistivity readings were made at approximately the same ground elevation (El. 560±) in both areas; however, groundwater levels were approximately 3 ft higher in the footing portion of the Auxiliary Building as indicated by the continuous seepage noted at approximate El. 545 through small fissures in the northern and northeastern portions of the Containment area excavation walls. On this basis, it is concluded that the decrease in average resistivity values obtained in the footing portion of the Auxiliary Building area is a result of the higher groundwater levels in portions of this area and is not an indication of significant solution activity in either building area.

Based on the preceding analysis, it is concluded that within the dimensional limits mentioned previously, the resistivity survey indicated uniform bedrock free of significant solution activity.

4. Conclusions:

The NX boring program, the geologic mapping program, and the rock probe program were completed in accordance with the requirements of the GQAM and the data obtained were analyzed.

Analysis of the data obtained from the boring program indicated that the percent core recovery averaged 98 percent, no zones of low drill time were penetrated, no sudden drops of the drill steel occurred, and no significant solution activity was observed on the rock cores recovered. Based on the results of these 51 NX borings and the results of 20 preconstruction NX borings made in the station area, it is concluded that no signficant solution activity was indicated by borings made in the station area.

Analysis of the data obtained from the geologic mapping program indicated that approximately 10 percent of the joints on the surface of the bedrock (El. 560±) were open and contained

indications of minor solution activity; however, the indication of minor solution activity did not extend below a depth of 3 ft beneath the bedrock surface. Joints mapped on excavated bedrock surfaces (including joints mapped in the pre-split walls below a depth of 3 ft) were typically less than 0.05 ft wide and were tightly filled with satin spar gypsum. No significant continuous joint systems were observed with depth. Based on these observations, it is concluded that no significant solution activity was indicated by the geologic mapping program.

Analysis of the data obtained fro the rock probe program indicated that there were neither zones of low drill time penetrated nor sudden drops of the drill steel noted during drilling of the 114 rock probes in the station area. In general, the grout taken in the pier rock probes was less than 10 cu ft. Grout takes were greater than 10 cu ft in nine of the probe holes; however, it was determined that these grout takes were the result of grout loss to the dewatering system or to the open faces of rock excavations and were not a result of significant solution activity.

Based on analysis of the data obtained during drilling 114 rock probes, field observations during grouting of the 112 rock probes drilled at pier locations, and data obtained during the drilling of 86 preconstruction rock probes, it is concluded that neither the drilling nor the grouting of rock probes in the station area indicated significant solution activity.

The seismic velocity survey, the gravity survey, and the resistivity survey were also completed in accordance with the requirements of the GQAM and the data obtained were analyzed.

Based on data obtained in the seismic velocity survey and taking into account the general considerations for selection of anomalous geophysical measurements, it was concluded that a seismic velocity of less than 9000 ft/sec would be considered anomalous. No anomalous seismic velocities were obtained during the survey in the station area (see Figures 2C.5-29 through 2C.5-32).

Based on data obtained in the gravity survey and taking into account the general considerations for selection of anomalous geophysical measurements, it was concluded that for topographically corrected Bouguer gravity readings, a gradient greater than 0.0015 milligals/ft between adjacent readings would be considered anomalous.

It has been shown that the results of the gravity survey indicated: (1) iso-gal contours drawn prior to the application of topographic correction follow the effects of the station area topography and no irregular low gravity areas are indicated (see Figure 2C.5-34); (2) the maximum difference in topographically corrected Bouguer gravity across the station area is only 0.060 milligals; and (3) iso-gal contours drawn after application of topographic corrections result in gravity gradients of less than 0.0015 milligals/ft between adjacent readings (see Figure 2C.5-36).

It is concluded that the small variations in topographically corrected Bouguer gravity readings (see Figure 2C5-36) across the station area are a result of regional gravity gradients not associated with significant solution in the bedrock.

Based on data obtained in the resistivity survey and taking into account the general considerations for the selection of anomalous geophysical measurements, it was concluded that in areas more than 20 ft from (a) the perimeter dewatering system (except in the northeastern corner of the Turbine Building area where this distance is increased to 40 ft), or (b) the faces of rock excavations in the Containment and Auxiliary Building area, a resistivity reading would be considered anomalous if it were more than 5000 ohm-cm above or below an average resistivity gradient determined for a particular foundation area. It has been shown that within the

dimensional limits mentioned previously, the resistivity values are within 5000 ohm-cm of the average resistivity gradient for particular foundation areas; and, therefore, no anomalous resistivity values were obtained which would be indicative of significant solution activity.

It is concluded that the variation in resistivity measurements in the station area are a result of the effects of topographic, groundwater, or geologic conditions that are not related to significant solution activity in the bedrock.

Based on (1) the absence of evidence of significant solution activity in 71 borings and 198 rock probes made in the station area prior to the beginning of and during construction; (2) the absence of the observation of significant solution activity on bedrock surfaces mapped during construction; and (3) the absence of anomalous geophysical measurements, it is concluded that there was no significant solution activity detected during part A of the Bedrock Verification Program. Therefore, implementation of part B of the program (i.e., exploration of significant solution activity found during part A) was not required. In addition, because no significant solution activity was detected, it is not necessary to implement any portion of the Remedial Treatment Program.

It is further concluded, based on the results of the GQAP, that the bedrock beneath the foundation of the power station has an allowable net bearing pressure, under static and dynamic loads, of 100 k/sq ft.

2C.5.3 Groundwater Monitoring Program

1. General:

The bedrock in the station area is susceptible to solution activity by flowing groundwater. For this reason, the dewatering system was designed to minimize both concentration of groundwater flow and groundwater flow in the bedrock immediately beneath structures; and a grout curtain wall was constructed outside of the clewatering system perimeter to minimize total groundwater flow.

Because operation of the construction dewatering system would cause water to flow through the bedrock, a groundwater monitoring program was to determine if significant solution activity was being caused in the bedrock by dewatering operations. The program consisted of making sulfate content, electrical conductivity, and dewatering system discharge measurements. The results of the program are presented in the following text.

2. Summary of the Requirements of the Groundwater Monitoring Program:

To assure that significant solution activity was not occurring as a result of construction dewatering operations, the sulfate content and the electrical conductivity of groundwater samples obtained from the dewatering system discharge pipe and from the seep on the north wall of the Containment area excavation were to be monitored on a periodic basis. The results of this monitoring program were to be compared with water analysis data obtained prior to dewatering operations. In addition to field testing, at periodic intervals during construction, water samples were to be sent to an off-site laboratory for more complete chemical analysis. Results of the field laboratory tests were then to be compared with the results of the off-site laboratory chemical analysis to verify the field laboratory results.

A judgement decision concerning the possibility of solution activity being caused by dewatering operations would be based on a comparison of the verified field laboratory results with preconstruction water analysis test results.

In addition to making chemical analysis and conductivity tests on groundwater samples, periodic measurements were to be made of the dewatering system discharge rate.

3. Presentation and Analysis of Results Obtained During Groundwater Monitoring Program:

Electrical conductivity measurements were made between 6 June 1970 and 20 November 1970; sulfate content determinations were made between 2 July 1970 and August 1972 and the dewatering system discharge rate was measured between 13 May 1970 and August 1972.

Electrical conductivity determination is an indirect measure of total dissolved solids in the water. An increase in the conductivity readings with time would result if the total dissolved solids in the groundwater were increased such an increase would indicate that the flowing groundwater was causing solution activity.

The results of the electrical conductivity measurements indicated that the conductivity of samples obtained from the dewatering system discharge pipe were always greater and were more erratic on a day-to-day basis than those of samples obtained from the seep in the Containment area.

Investigation of the cause of the difference in measured conductivity between the groundwater samples obtained from the seep in the Containment area and groundwater samples obtained from the dewatering system discharge pipe, indicated that the high and erratic conductivity readings of the latter were caused by the addition of acidizing agents to the wells (used to cleanse the well screens) and the chemical reaction of the corrosive groundwater on the metallic components of the dewatering system. Consequently, only electrical conductivity measurements made on samples obtained from the seep in the Containment area wall are considered meaningful. These conductivity measurements (see Figure 2C.5-40) indicate that the groundwater conductivity decreased slightly with time. Because of this slight decrease in conductivity, it is concluded that the total dissolved solids of the groundwater has not increased and no significant solution activity has resulted from the flow of water to the dewatering system.

The sulfate content determination is an approximate measure of the amount of gypsum ($CaSO_4$) dissolved in the groundwater. A large increase in sulfate content with time would result from an increase in the amount of dissolved gypsum in the groundwater; such an increase would indicate that the flowing groundwater was causing solution activity. The results of sulfate determinations made prior to construction and during the period 2 July 1970 through 29 November 1970 are presented in Figure 2C.5-41. Intermittent sulfate determinations were made by Pittsburgh Testing Laboratory between 20 November 1970 and August 1972.

Analysis of the results of the sulfate content determinations by both WMAI and PTL indicates that: (1) the preconstruction sulfate concentration ranged from 1300 to 1700 parts/million with the average of six determinations being 1550 parts/million; (2) the sulfate concentration determined between 2 July 1970 and August 1972 ranged from 1140 to 2000 part/million with the average of 89 determinations being approximately 1600 parts/million; and (3) the sulfate content determined during construction by off-site chemical analysis ranged from 1300 to 1950 parts/million with the average of four determinations being 1675 parts/million.

Based on the results of analysis of sulfate content data, it is concluded that there was no significant increase in the sulfate content of the groundwater as it flowed through the bedrock. Consequently, it is concluded that no significant solution activity resulted from operation of the dewatering system.

Calculations of the gypsum deficiency of the groundwater were made using a method described by Zverev (1967). The calculations indicate that the groundwater is and has been saturated with gypsum and, therefore, has little or no tendency to dissolve additional gypsum.

A large increase in the dewatering system discharge rate could indicate an increase in the size of voids in the bedrock or possibly a breaching of the grout curtain wall, or both. Such an increase in the size of voids or breaching of the grout curtain wall could result from the solution of bedrock caused by the flowing groundwater. The results of dewatering system discharge measurements are presented in Figure 2C.5-42.

The results of the dewatering system discharge rate measurements indicate that the dewatering system discharge rate remained between 300 and 350 gal/min between May 1970 and August 1972 (except for short duration increases of up to a maximum of 780 gal/min). The short duration increases resulted from the discharge of surface drainage from sumps into the dewatering discharge system when heavy rainfalls occurred. The stabilized discharge rate of 300 to 350 gal/min is equivalent to a flow of 0.15 to 0.17 gal/min/linear ft around the perimeter of the excavation. It is concluded that no significant solution activity is indicated by the results of the dewatering system discharge rate measurements.

4. Conclusions:

The analysis of the data obtained during the groundwater monitoring program indicated that: (1) the total dissolved solids (as determined using the measured electrical conductivity) decreased slightly between July and November 1970; (2) the average sulfate content of the groundwater measured intermittently throughout construction was approximately the same as that determined from measurements made prior to construction; and (3) the discharge rate of the construction dewatering system remained constant at approximately 350 gal/min.

If dewatering operations were causing solution of the bedrock: (1) the electrical conductivity and sulfate content of the water would increase, and (2) the discharge rate of the dewatering system would increase. However, because neither occurred, it is concluded that dewatering operations has not caused significant solution activity of the bedrock.

2C.5.4 Class I Earthwork Quality Assurance Program

1. General:

A Class I earthwork quality assurance program was implemented to inspect and test approximately 71,000 cu yd of fill placed and compacted for construction of the Class I dikes in the Intake Forebay area during the period 26 August through 6 October 1970. Location of the Class I dikes is presented in Figure 2C.5-43. Fill was placed and compacted in accordance with pertinent project specifications and project design drawings.

2. Summary of the Requirements of the Class I Earthwork Quality Assurance Program:

Continuous inspection was to be provided by an Inspector during the construction of Class I dikes. The Inspector was to observe the work of the earthwork contractor for the purpose of determining compliance with pertinent requirements of the project specifications.

In addition to observing the work of the earthwork contractor, the Inspector was to obtain water content samples in accordance with the project specifications and at locations determined on the basis of the Inspector's judgment. Twelve field vane shear strength tests were to be made in the Class I dike fill in order to assure that the preconstruction minimum design shear strength of soil was obtained using the specified compaction procedure.

3. Presentation and Analysis of Results Obtained During Class I Quality Assurance Program:

a. Field inspection:

Continuous field inspection of the contractor's operations during fill placement was provided by an Inspector during the period 26 August through 6 October 1970. The data obtained during this inspection were as follows: (1) fill load count; (2) uncompacted lift thickness; (3) compactor coverages; and (4) water content of the in-place fill and of the borrow area from which the fill was obtained. These data were recorded on summary data sheets for each day's operation and a daily fill placement plan was prepared. The daily fill placement plan presented: (1) the limits of fill placement during each work shift; (2) the elevation of the fill at the end of each shift; and (3) the limits of areas requiring remedial treatment (i.e., additional compaction, excavation of wet material, or removal of oversized rocks) in order to comply with the requirements specified in the contract documents.

A histogram of the water content data obtained from the compacted dike fill and from the borrow area is presented in Figure 2C.5-44.

Analysis of the data presented in this figure indicates that the water content data obtained from the compacted dike fill for each deposit ranged as follows:

Deposit	Maximum Value %	Minimum Value %
Glaciolacustrine (144 determinations)	36	18
Till (97 determinations)	22	9

Project specifications required that the water content of the in-place fill range between in-situ borrow area water content and optimum water content minus 5 percentage points. Results of laboratory tests made to define in-situ and optimum water content of fill material are discussed in the following text.

b. Laboratory testing:

1) Glaciolacustrine deposit:

Results of 577 water content tests on in-situ glaciolacustrine deposit samples obtained from on-site borrow areas indicated the in-situ water content of this material ranged from 16 to 37 percent with an average[1] of 24 percent (see Figure 2C.5-44).

Results of 17 laboratory compaction on samples of glaciolacustrine deposit indicated the optimum water content ranged from 18 to 24 percent with an average of 21 percent. Generally, the optimum water content was equal to, or slightly less than, the in-situ water content. Range and average value of maximum density and Atterberg limit values based on the laboratory tests are as follows:

Property	No. Determinations	Range of Values Maximum	Range of Values Minimum	Average Value
Maximum Dry Density, lb/cu ft	17	111.0	98.7	105.0
Liquid Limit, L_L	5	55	42	48
Plastic Limit, P_L	5	24	19	22

It was concluded, on the basis of these data, that glaciolacustrine deposit would be acceptable as Class I fill for water contents between 13 and 37 percent if on the basis of the Inspector's observations, the water content of the material was neither raised by ponded surface runoff water and groundwater nor lowered more than approximately 5 percentage points by drying.

2) Till deposit:

Results of 361 water content tests on in-situ till deposit samples, obtained from on-site borrow areas, indicated the in-situ water content of this material ranged from 9 to 21 percent with an average of 15 percent (see Figure 2C.5-44).

Results of seven laboratory compaction tests on samples of till deposit indicated the optimum water content ranged from 14 to 17 percent with average of 16 percent. Generally, the optimum water content was equal to, or slightly greater than, the in-situ water content. Range and average values of maximum density and Atterberg limit values based on the tests are as follows:

Property	No. Determinations	Range of Values Maximum	Range of Values Minimum	Average Value
Maximum Dry Density, lb/cu ft	7	119.6	113.0	115.0
Liquid Limit, L_L	2	32	29	30
Plastic Limit, P_L	2	18	16	17

[1] Average values in this report are arithmetic averages.

It was concluded, on the basis of these data, that till deposit would be acceptable as Class I fill for water contents between 9 and 21 percent if, on the basis of the Inspector's observations, the water content of the material was neither raised by ponded surface runoff water and groundwater nor lowered more than approximately 5 percentage points by drying.

C. Vane shear strength tests:

Thirteen in-situ vane shear strength tests were made at four locations within the Class I dikes to assure that a minimum design shear strength of 0.45 k/sq ft was obtained by the specified compaction procedure. The locations at which the tests were made are presented in Figure 2C.5-43. The tests were made between 16 September and 4 November 1970 using a 2-in.-dia, 5-in.-long Acker vane shear device.

The data obtained during the vane shear strength tests included: (1) the maximum torque required to shear the in-situ fill material; (2) the torque required to shear the in-situ fill material in a remolded state; (3) the water content of the fill at the depth the vane shear test was made; and (4) classification of the fill material on which the test was made.

The results of the vane shear strength tests are presented in Table 2C.5-9, 2C.5-10 and 2C.5-11. Analysis of the data indicates that the in-situ vane shear strengths obtained ranged from 1.7 to 7.0 k/sq ft, with the average being 4.6 k/sq ft; the remolded vane shear strength obtained ranged from 0.9 to 1.4 k/sq ft, with the average being 1.1 k/sq ft. Both the minimum in-situ and minimum remolded shear strengths obtained were greater than the required minimum design shear strength of the dike fill.

4. Conclusions:

Analysis of the data obtained from observations and tests made during inspection of Class I earthwork construction during the period 26 August through 6 October 1970 and analysis of results of 13 vane shear strength tests had indicated: (1) the uncompacted lift thickness of Class I fill and number of coverages of approved compactors over each lift met the requirements of the project specifications; (2) Class I fill was placed within the required water content range; and (3) the minimum vane shear strength determined for the in-situ Class I dike fill was greater than the minimum design shear strength.

Based on this analysis, it is concluded that Class I dike fill placed during the period 26 August through 6 October 1970 was constructed in accordance with project specifications. A discussion of stability of the Class I dikes is presented in Section 2C.6 of this Appendix.

Subsequent to 6 October 1970, additional Class I fill was placed and compacted. Inspection and testing of fill placement and compaction was by others.

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TABLE 2C.5-1
NX Boring Summary

	Coordinates		Orientation		Date (1970)		Elevation[(1)]	Elevation[(1)]	Significant
Boring No.	North ft	East ft	Angle [(2)] deg	Bearing deg	Started	Finished	Top of Boring ft	Bottom of Boring ft	Solution Activity Penetration
7-1	10,236	10,281	0	--	26 May	28 May	561.0	469.9	None
7-2	10,248	10,298	0	--	28 May	28 May	560.8	471.2	None
7-3	10,275	10,340	0	--	29 May	1 June	560.8	470.2	None
7-4	10,350	10,314	0	--	1 June	2 June	560.0	468.4	None
7-5	10,310	10,267	0	--	2 June	5 June	560.1	469.9	None
7-6	10,322	10,176	0	--	4 June	7 June	559.3	469.3	None
7-7	10,259	10,133	0	--	4 June	5 June	560.0	470.7	None
7-8	10,281	10,102	0	--	6 June	8 June	559.6	469.5	None
7-9	10,258	10,081	0	--	6 June	9 June	559.5	466.5	None
7-10	10,251	10,033	0	--	8 June	9 June	558.0	468.3	None
7-11	10,308	10,062	0	--	8 June	10 June	558.3	468.4	None
7-12	10,355	10,303	20	S14W	8 June	11 June	560.0	469.9	None
7-13	10,284	10,031	0	--	10 June	11 June	560.6	469.2	None
7-14	10,221	10,140	0	--	10 June	11 June	560.1	469.9	None
7-15	10,368	10,123	0	--	11 June	12 June	557.6	468.2	None
7-16	10,346	10,248	20	N48W	11 June	12 June	559.3	469.2	None
7-17	10,393	10,102	0	--	11 June	12 June	555.8	465.9	None
7-18	10,409	10,119	0	--	12 June	13 June	556.5	468.0	None
7-19	10,406	10,254	0	--	13 June	15 June	559.0	468.5	None
7-20	10,244	10,139	20	N45W	13 June	16 June	560.0	470.0	None
7-21	10,190	10,156	0	--	13 June	16 June	560.0	470.5	None
7-22	10,251	10,219	0	--	16 June	17 June	559.5	469.4	None
7-23	10,220	10,164	20	S46W	17 June	18 June	559.4	469.1	None
7-24	10,212	10,200	0	--	17 June	18 June	559.9	470.0	None
7-25	10,401	10,299	0	--	19 June	23 June	559.9	465.8	None
7-26	10,259	10,385	0	--	20 June	22 June	560.0	470.4	None
7-27	10,451	10,349	0	--	24 June	25 June	559.0	469.1	None
7-28	10,359	10,390	20	S45W	2 July	6 July	560.0	469.3	None
7-29	10,313	10,359	20	N90W	25 June	29 June	560.1	469.5	None

TABLE 2C.5-1 (Continued)
NX Boring Summary

	Coordinates		Orientation		Date (1970)		Elevation(1) Top of Boring	Elevation(1) Bottom of Boring	Significant Solution Activity Penetration
Boring No.	North ft	East ft	Angle (2) deg	Bearing deg	Started	Finished	ft	ft	
7-30	10,395	10,075	20	N45W	3 Sept	11 Sept	538.0	382.8	None
7-31	10,350	10,066	20	N90W	10 Sept	11 Sept	532.0	466.2	None
7-32	10,381	10,155	20	S10W	5 Sept	9 Sept	538.9	466.6	None
7-33	10,334	10,100	20	N45E	9 Sept	10 Sept	528.8	467.6	None
7-34	10,316	10,213	20	S45W	10 Aug	14 Aug	541.1	390.1	None
7-35	10,267	10,018	20	N90E	1 Sept	4 Sept	542.0	466.7	None
7-36	10,295	10,165	20	N45W	12 Sept	15 Sept	540.0	469.0	None
7-37	10,204	10,014	20	N45E	17 July	22 July	561.4	472.9	None
7-38	10,446	10,152	20	S45E	14 July	16 July	555.4	465.4	None
7-39	10,400	10,200	20	N45E	7 July	13 July	557.8	467.4	None
7-40	10,450	10,200	20	S45W	7 July	13 July	556.3	466.2	None
7-41	10,446	10,102	20	S55E	17 July	22 July	556.0	465.0	None
7-42	10,200	10,067	20	N45W	22 July	24 July	561.0	471.0	None
7-43	10,264	10,579	20	N45E	6 Aug	10 Aug	575.3	471.1	None
7-44	10,264	10,639	20	N45W	3 Aug	5 Aug	574.1	469.9	None
7-45	10,385	10,579	20	S45E	31 July	4 Aug	573.1	472.2	None
7-46	10,385	10,639	20	S45W	24 July	28 July	571.7	472.7	None
7-47	10,325	10,639	20	N90W	29 July	2 Aug	571.0	467.0	None
7-48	10,325	10,579	20	N90W	22 July	28 July	569.0	389.3	None
7-49	10,215	10,365	20	N45W	28 June	1 July	560.0	469.6	None
7-50	10,450	10,251	20	S45E	13 July	17 July	557.4	467.1	None
7-51	10,380	10,019	0	-	19 Aug	27 Aug	555.8	389.8	None

Notes: (1) Elevation referenced to International Great Lakes Datum (IGLD)
(2) Angle measured from the vertical

TABLE 2C.5-2
Foundation Grade Rock Probe Summary

Rock Probe No.	Coordinates North ft	Coordinates East ft	Date (1970) Started	Date (1970) Finished	Orientation Angle (1) deg	Orientation Bearing deg	Elevation (2) Top of Probe ft	Elevation (2) Bottom of Probe ft	Significant Solution Activity Indicated by Probe
RPF-1	10,355	10,245	31 July	31 July	20	5	560	540	None
RPF-2	10,225	10,065	31 July	31 July	20	5	560	540	None

Notes: (1) Angle measured from the vertical
(2) Elevation referenced to International Great Lakes Datum (IGLD)

TABLE 2C.5-3

Pier Probe Summary - Drilling

	Coordinates (1)		Pier Dimensions (1)				
Pier Mark	North ft	East ft	Diameter (3) ft	Depth ft	Probe (2) Depth ft	Date Drilled	Significant Solution Activity Indicated by Drilling
Ab-1*	10468.0	10351.7	5.0	7.5	17.5	15 May 70	none
Ab-2	10442.5	10351.7	5.0	5.0	15.0	15 Sept 70	none
Ab-3	10417.5	10351.7	5.0	5.0	15.0	15 Sept 70	none
Ab-4	10392.5	10351.7	5.0	5.0	15.0	15 Sept 70	none
BBa(a)-4d	10368.0	10352.3	7.0 x 14.1	8.0	30.5	1 Feb 71	none
BBa(a)-6	10333.5	10352.3	7.0 x 14.1	8.0	30.5	1 Feb 71	none
BBa(a)-7a	10299.0	10352.3	7.0 x 14.1	8.0	30.0	2 Feb 71	none
Aa-8	10274.5	10352.8	5.0	5.0	15.0	16 Sept 70	none
Aa-9	10249.5	10352.8	5.0	6.0	15.0	16 Sept 70	none
Aa-10	10221.5	10352.8	5.0	6.0	15.0	16 Sept 70	none
Aa-11a	10196.0	10352.8	4.0 x 6.0	6.0	18.0	14 Sept 70	none
BBa(b)-4d	10368.0	10360.5	7.0 x 14.1	8.0	32.5	1 Feb 71	none
BBa(b)-6	10333.5	10360.5	7.0 x 14.1	8.0	32.5	1 Feb 71	none
BBa(b)-7a	10299.0	10360.5	7.0 x 14.1	8.0	30.0	2 Feb 71	none
B-7	10299.5	10328.0	3.5	4.0	15.5	15 Sept 70	none
B-8	10274.5	10328.0	4.0	4.5	12.5	16 Sept 70	none
B-9	10249.5	10328.0	4.0	4.5	12.5	16 Sept 70	none
B-10	10221.5	10328.0	3.5	4.0	10.0	16 Sept 70	none
B-11a	10196.0	10328.0	5.0	7.5	17.5	14 Sept 70	none
Ba-1*	10468.0	10321.0	5.0	7.5	17.5	15 Sept 70	none
Ba-2	10442.5	10321.0	3.0	3.0	9.0	15 Sept 70	none
Ba-3	10417.5	10321.0	3.0	3.0	9.0	15 Sept 70	none
Ba-4	10392.5	10321.0	3.0	3.0	9.0	15 Sept 70	none

Notes: (1) Coordinates and pier dimensions taken from Bechtel Drawing C-65 Rev 6 dated 5 Jan 1971
(2) Probe depth referenced to existing rock surface
(3) Piers marked BBa-4d, 6, and 7a and Aa-11a are rectangular – all other piers are circular
(4) Pier marks determined from lettered and numbered construction contol lines on Bechtel drawing C-65 Rev 6 dated 5 Jan 1971. Asterisk (*) indicates pier line located 6 in. north of construction control line no. 1

TABLE 2C.5-3 (Continued)
Pier Probe Summary - Drilling

Pier Mark	Coordinates [1] North ft	East ft	Pier Dimensions [1] Diameter [3] ft	Depth ft	Probe [2] Depth ft	Date Drilled	Significant Solution Activity Indicated by Drilling
Ba-5	10367.5	10321.0	3.5	4.0	15.5	15 Sept 70	none
Ba-6	10333.5	10321.0	3.5	4.0	16.0	15 Sept 70	none
BB-8a	10273.5	10373.0	3.0	3.0	9.0	16 Sept 70	none
BB-9a	10247.5	10373.0	3.0	3.0	9.0	16 Sept 70	none
BB-10	10221.5	10373.0	3.0	3.0	9.0	16 Sept 70	none
BB-11	10196.5	10373.0	4.5	5.0	14.0	14 Sept 70	none
Bb-9	10249.5	10303.9	4.0	4.5	12.5	16 Sept 70	none
C-1*	10468.0	10303.0	5.0	5.0	15.0	15 Sept 70	none
C-10	10221.5	10303.0	3.5	3.5	10.5	16 Sept 70	none
C-11a	10196.0	10303.0	5.0	7.5	17.5	14 Sept 70	none
CCa-4c	10368.5	10376.5	7.0	8.0	22.0	1 Feb 71	none
CCa-6	10333.5	10376.5	7.0	8.0	22.0	2 Feb 71	none
CCa-7b	10298.5	10375.5	7.0	8.0	25.0	1 Feb 71	none
CC-4c	10368.5	10393.0	7.0	8.0	23.0	1 Feb 71	none
CC-6	10333.5	10393.0	7.0	8.0	23.0	1 Feb 71	none
CC-7b	10298.5	10393.0	7.0	8.0	25.0	1 Feb 71	none
CC-8a	10273.5	10393.0	3.0	3.0	9.0	15 Sept 70	none
CC-9a	10247.5	10393.0	3.0	3.0	9.0	15 Sept 70	none
CC-10	10221.5	10393.0	3.0	3.0	9.0	15 Sept 70	none
CC-11	10196.5	10393.0	4.5	5.0	14.0	14 Sept 70	none
D-1*	10468.0	10278.0	5.0	7.5	17.5	15 Sept 70	none
Da-9	10249.5	10277.1	4.0	4.5	12.5	15 Sept 70	none

Notes: (1) Coordinates and pier dimensions taken from Bechtel Drawing C-65 Rev 6 dated 5 Jan 1971
(2) Probe depth referenced to existing rock surface
(3) Piers marked BBa-4d, 6, and 7a and Aa-11a are rectangular – all other piers are circular
(4) Pier marks determined from lettered and numbered construction contol lines on Bechtel drawing C-65 Rev 6 dated 5 Jan 1971. Asteris (*) indicates pier line located 6 in. north of construction control line no. 1.

TABLE 2C.5-3 (Continued)

Pier Probe Summary - Drilling

Pier Mark	Coordinates (1) North ft	East ft	Pier Dimensions (1) Diameter (3) ft	Depth ft	Probe (2) Depth ft	Date Drilled	Significant Solution Activity Indicated by Drilling
D-10	10221.5	10278.0	4.0	4.5	12.5	14 Sept 70	none
D-11a	10196.0	10278.0	5.0	7.5	17.5	14 Sept 70	none
E-1*	10468.0	10253.0	5.0	5.0	15.0	15 Sept 70	none
E-2	10442.5	10253.0	3.5	4.0	11.0	15 Sept 70	none
E-3	10417.5	10253.0	3.5	4.0	10.0	15 Sept 70	none
E-4	10392.5	10253.0	3.5	4.0	10.0	15 Sept 70	none
E-5	10367.5	10253.0	3.5	4.5	11.5	15 Sept 70	none
E-6	10333.5	10253.0	3.5	4.5	11.5	15 Sept 70	none
E-7	10299.5	10253.0	3.5	4.5	11.5	14 Sept 70	none
E-8	10274.5	10253.0	4.0	4.5	13.0	14 Sept 70	none
E-9	10249.5	10253.0	4.0	4.5	13.0	14 Sept 70	none
E-10	10221.5	10253.0	3.5	3.5	10.5	14 Sept 70	none
E-11a	10196.0	10253.0	5.0	7.5	17.5	14 Sept 70	none
Ea-1*	10468.0	10229.3	5.0	7.5	17.5	19 Sept 70	none
Eb-2	10442.5	10229.0	5.0	5.0	15.0	16 Sept 70	none
Eb-3	10417.5	10229.0	5.0	5.0	15.0	16 Sept 70	none
Eb-4	10392.5	10229.0	5.0	5.0	15.0	16 Sept 70	none
Eb-5	10367.5	10229.0	5.0	7.5	17.5	24 Sept 70	none
Eb-6	10333.5	10229.0	5.0*	6.0	18.0	24 Sept 70	none
Eb-7	10299.5	10229.0	5.0	7.5	17.5	17 Sept 70	none
Eb-8	10274.5	10229.0	5.0	7.5	17.5	17 Sept 70	none
Eb-9	10249.5	10229.0	5.0	7.5	17.5	17 Sept 70	none

Notes: (1) Coordinates and pier dimensions taken from Bechtel Drawing C-65 Rev 6 dated 5 Jan 1971
(2) Probe depth referenced to existing rock surface
(3) Piers marked BBa-4d, 6, and 7a and Aa-11a are rectangular – all other piers are circular
(4) Pier marks determined from lettered and numbered construction contol lines on Bechtel drawing C-65 Rev 6 dated 5 Jan 1971. Asteris (*) indicates pier line located 6 in. north of construction control line no. 1.

TABLE 2C.5-3 (Continued)
Pier Probe Summary - Drilling

Pier Mark	Coordinates (1) North ft	East ft	Pier Dimensions (1) Diameter (3) ft	Depth ft	Probe (2) Depth ft	Date Drilled	Significant Solution Activity Indicated by Drilling
Eb-10	10221.5	10229.0	5.0	7.5	17.5	17 Sept 70	none
Ea-11a	10196.0	10229.3	5.0	7.5	17.5	17 Sept 70	none
Fa-1a	10466.2	10224.2	3.5	2.5	9.5	16 Sept 70	none
Fb-1a	10466.2	10208.5	3.5	2.5	9.5	24 Sept 70	none
Fc-1a	10466.2	10192.5	3.5	2.5	9.5	24 Sept 70	none
Ga-1a	10466.2	10177.5	3.5	2.5	9.5	24 Sept 70	none
Ge-1a	10466.2	10163.5	3.5	2.5	9.5	24 Sept 70	none
Gf-1a	10466.2	10148.0	3.5	2.5	9.5	24 Sept 70	none
Gg-1a	10466.2	10130.0	3.5	2.5	9.5	24 Sept 70	none
J-1a	10466.2	10100.0	3.5	2.5	9.5	24 Sept 70	none
Jb-1a	10466.2	10074.8	3.5	2.5	9.5	24 Sept 70	none
Gg-1b	10444.0	10130.0	3.5	2.5	9.5	24 Sept 70	none
J-1b	10444.0	10100.0	3.5	2.5	9.5	24 Sept 70	none
Jb-1b	10444.0	10074.8	3.5	2.5	9.5	24 Sept 70	none
Fa-2a	10438.8	10224.2	3.5	2.5	9.5	16 Sept 70	none
Fb-2a	10438.8	10208.5	3.5	2.5	9.5	16 Sept 70	none
Fc-2a	10438.8	10192.5	3.5	2.5	9.5	24 Sept 70	none
Ga-2a	10438.8	10177.5	3.5	2.5	9.5	16 Sept 70	none
Ge-2a	10438.8	10163.5	3.5	2.5	9.5	16 Sept 70	none
Gf-2a	10438.8	10148.0	3.5	2.5	9.5	16 Sept 70	none
Gg-2d	10422.0	10130.0	3.5	2.5	9.5	16 Sept 70	none
J-2b	10428.0	10100.0	3.5	2.5	9.5	16 Sept 70	none
Jb-2c	10424.0	10074.8	3.5	2.5	9.5	16 Sept 70	none

Notes: (1) Coordinates and pier dimensions taken from Bechtel Drawing C-65 Rev 6 dated 5 Jan 1971
(2) Probe depth referenced to existing rock surface
(3) Piers marked BBa-4d, 6, and 7a and Aa-11a are rectangular – all other piers are circular
(4) Pier marks determined from lettered and numbered construction contol lines on Bechtel drawing C-65 Rev 6 dated 5 Jan 1971. Asteris (*) indicates pier line located 6 in. north of construction control line no. 1.

TABLE 2C.5-3 (Continued)

Pier Probe Summary - Drilling

Pier Mark	Coordinates (1) North ft	East ft	Pier Dimensions (1) Diameter (3) ft	Depth ft	Probe (2) Depth ft	Date Drilled	Significant Solution Activity Indicated by Drilling
Fa-3a	10409.8	10224.2	3.5	2.5	9.5	16 Sept 70	none
Fb-3a	10409.8	10208.5	3.5	2.5	9.5	16 Sept 70	none
Fc-3a	10409.8	10192.5	3.5	2.5	9.5	16 Sept 70	none
Ga-3a	10409.8	10177.5	3.5	2.5	9.5	16 Sept 70	none
Ge-3a	10409.8	10163.5	3.5	2.5	9.5	16 Sept 70	none
Gf-3a	10409.8	10148.0	3.5	2.5	9.5	16 Sept 70	none
Fa-3b	10395.8	10224.2	3.5	2.5	9.5	16 Sept 70	none
Fb-3b	10395.8	10208.5	3.5	2.5	9.5	16 Sept 70	none
Fd-3b	10395.8	10187.2	3.5	2.5	9.5	16 Sept 70	none
Fd-4a	10385.5	10187.2	3.5	2.5	9.5	16 Sept 70	none
Gd-4a	10385.5	10168.5	3.5	2.5	9.5	16 Sept 70	none
Fd-4b	10378.5	10187.2	3.0	2.0	8.0	16 Sept 70	none
Fd-5a	10359.5	10187.2	3.0	2.0	8.0	24 Sept 70	none
Gb-5a	10359.5	10176.2	3.0	2.0	8.0	16 Sept 70	none
Ja-10a	10212.0	10075.0	2.5	3.0	8.0	17 Sept 70	none
Ja-10b	10207.0	10075.0	2.5	3.0	8.0	17 Sept 70	none
Jc-10a	10212.0	10058.0	2.5	3.0	8.0	17 Sept 70	none
Jc-10b	10207.0	10058.0	2.5	3.0	8.0	17 Sept 70	none
Jd-10a	10212.0	10041.0	2.5	3.0	8.0	17 Sept 70	none
Jd-10b	10207.0	10041.0	2.5	3.0	8.0	17 Sept 70	none
Je-10a	10212.0	10024.0	2.5	3.0	8.0	17 Sept 70	none
Je-10b	10207.0	10024.0	2.5	3.0	8.0	17 Sept 70	none

Notes: (1) Coordinates and pier dimensions taken from Bechtel Drawing C-65 Rev 6 dated 5 Jan 1971
(2) Probe depth referenced to existing rock surface
(3) Piers marked BBa-4d, 6, and 7a and Aa-11a are rectangular – all other piers are circular
(4) Pier marks determined from lettered and numbered construction contol lines on Bechtel drawing C-65 Rev 6 dated 5 Jan 1971. Asteris (*) indicates pier line located 6 in. north of construction control line no. 1.

TABLE 2C.5-4

Pier Probe Summary - Grouting

Pier Mark	Coordinates (1) North ft	Coordinates (1) East ft	Probe (2) Depth ft	Packer(2) Depth ft	Date Grouted	Grout Pressure lbs/in^2	Initial Grout Mix Water: Cement by Weight	Final Grout Mix Water: Cement by Weight	Total Grout Take ft^3	Remarks(3)	Significant Solution Activity Indicated by Grout Takes
Ab-1*	10468.0	10351.7	17.5	7.5	23 Sept 70	10	3:1	3:1	1.00	-	none
Ab-2	10442.5	10351.7	15.0	5.0	23 Sept 70	5	3:1	1:1	18.00	a	none
Ab-3	10417.5	10351.7	15.0	5.0	23 Sept 70	5	3:1	3:1	1.50	-	none
Ab-4	10392.5	10351.7	15.0	5.0	23 Sept 70	5	3:1	3:1	1.25	-	none
BBa(a)-4d	10368.0	10352.3	30.5	8.0	3 Feb 71	8	-	-	0.16	c	none
BBa(a)-6	10333.5	10352.3	30.5	8.0	3 Feb 71	8	-	-	0.07	c	none
BBa(a)-7a	10299.0	10352.3	30.0	8.0	3 Feb 71	8	-	-	0.07	c	none
Aa-8	10274.5	10352.8	15.0	5.0	23 Sept 70	5	3:1	3:1	1.00	-	none
Aa-9	10249.5	10352.8	15.0	5.0	23 Sept 70	5	3:1	1:1	17.00	a	none
Aa-10	10221.5	10352.8	15.0	5.0	16 Sept 70	5	3:1	1:1	18.00	a	none
Aa-11a	10196.0	10352.8	18.0	6.0	16 Sept 70	10	3:1	3:1	1.00	-	none
BBa(b)-4d	10368.0	10360.5	32.5	8.0	3 Feb 71	8	-	-	0.14	c	none
BBa(b)-6	10333.5	10360.5	32.5	8.0	3 Feb 71	8	-	-	0.14	c	none
BBa(b)-7a	10299.0	10360.5	30.5	8.0	3 Feb 71	8	-	-	0.14	c	none
B-7	10299.5	10328.0	15.5	4.0	23 Sept 70	5	1:1	1:1	7.00	-	none
B-8	10274.5	10328.0	12.5	4.5	23 Sept 70	5	3:1	3:1	2.50	-	none
B-9	10249.5	10328.0	12.5	4.5	23 Sept 70	5	3:1	3:1	1.50	-	none
B-10	10221.5	10328.0	10.0	4.0	16 Sept 70	5	3:1	3:1	2.00	-	none
B-11a	10196.0	10328.0	17.5	7.5	16 Sept 70	10	3:1	3:1	2.00	-	none

Notes: (1) Coordinates and pier dimensions taken from Bechtel Co. Drawing C-65 Rev 6 dated 5 Jan 1971.
(2) Depths referred to existing rock surface at pier location.
(3) Remark: (a) - Grout communication noted with dewatering system.
(b) - Grout leakage from fissures on vertical pre-split walls.
(c) - Probe water tested below bottom of pier at required grouting pressure. No significant take of water. No proof of grouting done. Probe hole backfilled with 1:1 grout mixture.
(4) Pier marks determined from lettered and numbered construction contol lines on Bechtel drawing C-65 Rev 6 dated 5 Jan 1971. Asteris (*) indicates pier line located 6 in. north of construction control line no. 1.

TABLE 2C.5-4 (Continued)

Pier Probe Summary – Grouting

Pier Mark	Coordinates (1) North ft	Coordinates (1) East ft	Probe (2) Depth ft	Packer(2) Depth ft	Date Grouted	Grout Pressure lbs/in²	Initial Grout Mix Water: Cement by Weight	Final Grout Mix Water: Cement by Weight	Total Grout Take ft³	Remarks(3)	Significant Solution Activity Indicated by Grout Takes
Ba-1*	10468.0	10321.0	17.5	5.0	16 Sept 70	5	3:1	3:1	1.50	-	none
Ba-2	10442.5	10321.0	9.0	3.0	23 Sept 70	5	3:1	3:1	0.50	-	none
Ba-3	10417.5	10321.0	9.0	3.0	23 Sept 70	5	3:1	3:1	0.50	-	none
Ba-4	10392.5	10321.0	9.0	3.0	23 Sept 70	5	3:1	3:1	1.00	-	none
Ba-5	10367.5	10321.0	15.5	4.0	23 Sept 70	5	3:1	3:1	1.00	-	none
Ba-6	10333.5	10321.0	16.0	4.5	23 Sept 70	5	3:1	3:1	8.00	-	none
BB-8a	10273.5	10373.0	9.0	3.0	23 Sept 70	5	3:1	3:1	1.00	-	none
BB-9a	10247.5	10373.0	9.0	3.0	22 Sept 70	5	3:1	3:1	0.50	-	none
BB-10	10221.5	10373.0	9.0	3.0	22 Sept 70	5	3:1	1:1	1.00	-	none
BB-11	10196.5	10373.0	14.0	5.0	16 Sept 70	5	3:1	3:1	0.25	-	none
Bb-9	10249.5	10303.9	12.5	4.0	16 Sept 70	5	3:1	3:1	0.50	-	none
C-1*	10468.0	10303.0	15.0	5.0	23 Sept 70	5	3:1	3:1	0.50	-	none
C-10	10221.5	10303.0	10.0	3.5	16 Sept 70	5	3:1	3:1	1.00	-	none
C-11a	10196.0	10303.0	17.5	7.5	16 Sept 70	10	3:1	3:1	0.75	-	none
CCa-4c	10368.5	10376.5	22.0	8.0	3 Feb 71	8	-	-	0.07	c	none
CCa-6	10333.5	10376.5	22.0	8.0	3 Feb 71	8	-	-	0.07	c	none
CCa-7b	10298.5	10375.5	25.0	8.0	3 Feb 71	8	-	-	1.00	c	none
CC-4c	10368.5	10393.0	23.0	8.0	3 Feb 71	8	-	-	0.07	c	none
CC-6	10333.5	10393.0	23.0	8.0	3 Feb 71	8	-	-	0.07	c	none
CC-7b	10298.5	10393.0	25.0	8.0	3 Feb 71	8	-	-	0.14	c	none

Notes: (1) Coordinates and pier dimensions taken from Bechtel Co. Drawing C-65 Rev 6 dated 5 Jan 1971.

(2) Depths referred to existing rock surface at pier location.

(3) Remark: (a) - Grout communication noted with dewatering system.

(b) - Grout leakage from fissures on vertical pre-split walls.

(c) - Probe water tested below bottom of pier at required grouting pressure. No significant take of water. No proof of grouting done. Probe hole backfilled with 1:1 grout mixture.

(4) Pier marks determined from lettered and numbered construction control lines on Bechtel drawing C-65 Rev 6 dated 5 Jan 1971. Asteris (*) indicates pier line located 6 in. north of construction control line no. 1.

TABLE 2C.5-4 (Continued)

Pier Probe Summary – Grouting

Pier Mark	Coordinates (1) North ft	East ft	Probe (2) Depth ft	Packer(2) Depth ft	Date Grouted	Grout Pressure lbs/in^2	Initial Grout Mix Water: Cement by Weight	Final Grout Mix Water: Cement by Weight	Total Grout Take ft^3	Remarks(3)	Significant Solution Activity Indicated by Grout Takes
CC-8a	10273.5	10393.0	9.0	3.0	23 Sept 70	5	3:1	3:1	2.00	-	none
CC-9a	10247.5	10393.0	9.0	4.5	23 Sept 70	5	3:1	3:1	1.00	-	none
CC-10	10221.5	10393.0	9.0	3.0	22 Sept 70	5	3:1	3:1	1.00	-	none
CC-11	10196.5	10393.0	14.0	5.0	16 Sept 70	5	3:1	3:1	0	-	none
D-1*	10468.0	10278.0	17.5	7.5	22 Sept 70	10	3:1	1:1	2.50	-	none
Da-9	10249.5	10277.1	12.5	4.5	23 Sept 70	5	3:1	3:1	2.25	-	none
D-10	10221.5	10278.0	12.5	4.5	16 Sept 70	5	3:1	3:1	1.00	-	none
D-11a	10196.0	10278.0	17.5	7.5	16 Sept 70	10	3:1	1:1	13.00	a	none
E-1*	10468.0	10253.0	15.0	5.0	22 Sept 70	5	3:1	1:1	2.50	-	none
E-2	10442.5	10253.0	11.0	4.0	16 Sept 70	5	3:1	1:1	1.50	-	none
E-3	10417.5	10253.0	10.0	4.0	23 Sept 70	5	3:1	3:1	1.00	-	none
E-4	10392.5	10253.0	10.0	4.0	21 Sept 70	5	3:1	3:1	0.50	-	none
E-5	10367.5	10253.0	11.5	4.5	21 Sept 70	5	3:1	1:1	0.25	-	none
E-6	10333.5	10253.0	11.5	4.5	21 Sept 70	5	3:1	3/4:1	28.00	b	none
E-7	10299.5	10253.0	11.5	4.5	21 Sept 70	5	3:1	3:1	2.75	-	none
E-8	10274.5	10253.0	13.0	4.5	21 Sept 70	5	3:1	3:1	1.50	-	none
E-9	10249.5	10253.0	13.0	4.5	21 Sept 70	5	3:1	3:1	1.00	-	none
E-10	10221.5	10253.0	10.5	3.5	21 Sept 70	5	3:1	1:1	17.00	b	none
E-11a	10196.0	10253.0	17.5	7.5	21 Sept 70	10	3:1	1:1	1.50	-	none

Notes: (1) Coordinates and pier dimensions taken from Bechtel Co. Drawing C-65 Rev 6 dated 5 Jan 1971.
(2) Depths referred to existing rock surface at pier location.
(3) Remark: (a) - Grout communication noted with dewatering system.
(b) - Grout leakage from fissures on vertical pre-split walls.
(c) - Probe water tested below bottom of pier at required grouting pressure. No significant take of water. No proof of grouting done. Probe hole backfilled with 1:1 grout mixture.
(4) Pier marks determined from lettered and numbered construction control lines on Bechtel drawing C-65 Rev 6 dated 5 Jan 1971. Asteris (*) indicates pier line located 6 in. north of construction control line no. 1.

TABLE 2C.5-4 (Continued)

Pier Probe Summary – Grouting

Pier Mark	Coordinates [(1)] North ft	Coordinates [(1)] East ft	Probe [(2)] Depth ft	Packer[(2)] Depth ft	Date Grouted	Grout Pressure lbs/in²	Initial Grout Mix Water: Cement by Weight	Final Grout Mix Water: Cement by Weight	Total Grout Take ft³	Remarks[(3)]	Significant Solution Activity Indicated by Grout Takes
Ea-1*	10468.0	10229.3	17.5	7.5	22 Sept 70	10	3:1	3:1	6.50	-	none
Eb-2	10442.5	10229.0	15.0	5.0	22 Sept 70	5	3:1	3:1	0.50	-	none
Eb-3	10417.5	10229.0	15.0	5.0	22 Sept 70	5	3:1	3:1	1.00	-	none
Eb-4	10392.5	10229.0	15.0	5.0	21 Sept 70	5	3:1	3:1	1.00	-	none
Eb-5	10367.5	10229.0	17.5	7.5	24 Sept 70	10	3:1	3:1	2.00	-	none
Eb-6	10333.5	10229.0	18.0	6.0	25 Sept 70	10	3:1	3:1	0.75	-	none
Eb-7	10299.5	10229.0	17.5	7.5	21 Sept 70	10	3:1	3:1	1.00	-	none
Eb-8	10274.5	10229.0	17.5	7.5	21 Sept 70	10	3:1	3:1	0.25	-	none
Eb-9	10249.5	10229.0	17.5	7.5	21 Sept 70	10	3:1	3:1	0.50	-	none
Eb-10	10221.5	10229.0	17.5	7.5	21 Sept 70	10	3:1	3:1	0.25	-	none
Ea-11a	10196.0	10229.3	17.5	7.5	21 Sept 70	10	3:1	3:1	0	-	none
Fa-1a	10466.2	10224.2	9.5	2.5	22 Sept 70	5	3:1	3:1	1.25	-	none
Fb-1a	10466.2	10208.5	9.5	2.5	24 Sept 70	5	3:1	3:1	5.00	a	none
Fc-1a	10466.2	10192.5	9.5	2.5	24 Sept 70	5	3:1	3:1	3.00	-	none
Ga-1a	10466.2	10177.5	9.5	2.5	24 Sept 70	5	3:1	3:1	9.00	a	none
Ge-1a	10466.2	10163.5	9.5	2.5	24 Sept 70	5	3:1	3:1	1.50	-	none
Gf-1a	10466.2	10148.0	9.5	2.5	24 Sept 70	5	3:1	3:1	1.00	-	none
Gg-1a	10466.2	10130.0	9.5	2.5	24 Sept 70	5	3:1	1:1	1.00	-	none
J-1a	10466.2	10100.0	9.5	2.5	25 Sept 70	5	3:1	3:1	0.75	-	none
Jb-1a	10466.2	10074.8	9.5	2.5	25 Sept 70	5	3:1	3:1	2.50	-	none

Notes: (1) Coordinates and pier dimensions taken from Bechtel Co. Drawing C-65 Rev 6 dated 5 Jan 1971.
(2) Depths referred to existing rock surface at pier location.
(3) Remark: (a) - Grout communication noted with dewatering system.
(b) - Grout leakage from fissures on vertical pre-split walls.
(c) - Probe water tested below bottom of pier at required grouting pressure. No significant take of water. No proof of grouting done. Probe hole backfilled with 1:1 grout mixture.
(4) Pier marks determined from lettered and numbered construction control lines on Bechtel drawing C-65 Rev 6 dated 5 Jan 1971. Asteris (*) indicates pier line located 6 in. north of construction control line no. 1.

TABLE 2C.5-4 (Continued)

Pier Probe Summary – Grouting

Pier Mark	Coordinates (1) North ft	East ft	Probe (2) Depth ft	Packer(2) Depth ft	Date Grouted	Grout Pressure lbs/in^2	Initial Grout Mix Water: Cement by Weight	Final Grout Mix Water: Cement by Weight	Total Grout Take ft^3	Remarks(3)	Significant Solution Activity Indicated by Grout Takes
Gg-1b	10444.0	10130.0	9.5	2.5	24 Sept 70	5	3:1	3:1	1.25	-	none
J-1b	10444.0	10100.0	9.5	2.5	25 Sept 70	5	3:1	3:1	0.25	-	none
Jb-1b	10444.0	10074.8	9.5	2.5	25 Sept 70	5	3:1	3:1	2.25	-	none
Fa-2a	10438.8	10224.2	9.5	2.5	22 Sept 70	5	3:1	1:1	1.00	-	none
Fb-2a	10438.8	10208.5	9.5	2.5	22 Sept 70	5	3:1	1:1	1.00	-	none
Fc-2a	10438.8	10192.5	9.5	2.5	25 Sept 70	5	3:1	1:1	6.25	a	none
Ga-2a	10438.8	10177.5	9.5	2.5	22 Sept 70	5	3:1	1:1	1.00	-	none
Ge-2a	10438.8	10163.5	9.5	2.5	21 Sept 70	5	3:1	3:1	5.25	-	none
Gf-2a	10438.8	10148.0	9.5	2.5	25 Sept 70	5	3:1	3:1	3.50	-	none
Gg-2d	10422.0	10130.0	9.5	2.5	17 Sept 70	5	3:1	3:1	1.00	-	none
J-2b	10428.0	10100.0	9.5	2.5	17 Sept 70	5	3:1	3:1	0.50	-	none
Jb-2c	10424.0	10074.8	9.5	2.5	17 Sept 70	5	3:1	3:1	9.00	-	none
Fa-3a	10409.8	10224.2	9.5	2.5	22 Sept 70	5	3:1	1:1	1.00	-	none
Fb-3a	10409.8	10208.5	9.5	2.5	21 Sept 70	5	3:1	3:1	1.25	-	none
Fc-3a	10409.8	10192.5	9.5	2.5	21 Sept 70	5	3:1	3:1	0.50	-	none
Ga-3a	10409.8	10177.5	9.5	2.5	21 Sept 70	5	3:1	3:1	0.75	-	none
Ge-3a	10409.8	10163.5	9.5	2.5	22 Sept 70	5	3:1	1:1	1.25	-	none
Gf-3a	10409.8	10148.0	9.5	2.5	17 Sept 70	5	3:1	3:1	0	-	none
Fa-3b	10395.8	10224.2	9.5	2.5	21 Sept 70	5	3:1	3:1	0.75	-	none
Fb-3b	10395.8	10208.5	9.5	2.5	21 Sept 70	5	3:1	3:1	1.75	-	none

Notes: (1) Coordinates and pier dimensions taken from Bechtel Co. Drawing C-65 Rev 6 dated 5 Jan 1971.

(2) Depths referred to existing rock surface at pier location.

(3) Remark: (a) - Grout communication noted with dewatering system.

(b) - Grout leakage from fissures on vertical pre-split walls.

(c) - Probe water tested below bottom of pier at required grouting pressure. No significant take of water. No proof of grouting done. Probe hole backfilled with 1:1 grout mixture.

(4) Pier marks determined from lettered and numbered construction control lines on Bechtel drawing C-65 Rev 6 dated 5 Jan 1971. Asteris (*) indicates pier line located 6 in. north of construction control line no. 1.

TABLE 2C.5-4 (Continued)

Pier Probe Summary – Grouting

Pier Mark	Coordinates (1) North ft	Coordinates (1) East ft	Probe (2) Depth ft	Packer(2) Depth ft	Date Grouted	Grout Pressure lbs/in^2	Initial Grout Mix Water: Cement by Weight	Final Grout Mix Water: Cement by Weight	Total Grout Take ft^3	Remarks(3)	Significant Solution Activity Indicated by Grout Takes
Fd-3b	10395.8	10187.2	9.5	2.5	22 Sept 70	5	3:1	1:1	1.00	-	none
Fd-4a	10385.5	10187.2	9.5	2.5	22 Sept 70	5	3:1	1:1	1.00	-	none
Gd-4a	10385.5	10168.5	9.5	2.5	17 Sept 70	5	3:1	3:1	1.00	-	none
Fd-4b	10378.5	10187.2	8.0	2.0	22 Sept 70	5	3:1	1:1	1.25	-	none
Fd-5a	10359.5	10187.2	8.0	2.0	24 Sept 70	5	3:1	1:1	12.00	b	none
Gb-5a	10359.5	10176.2	8.0	2.0	17 Sept 70	5	3:1	3:1	1.00	-	none
Ja-10a	10212.0	10075.0	8.0	3.0	24 Sept 70	5	3:1	1:1	13.00	b	none
Ja-10b	10207.0	10075.0	8.0	3.0	24 Sept 70	5	3:1	3:1	4.50	-	none
Jc-10a	10212.0	10058.0	8.0	3.0	24 Sept 70	5	3:1	3:1	5.00	-	none
Jc-10b	10207.0	10058.0	8.0	3.0	24 Sept 70	5	3:1	3:1	1.00	-	none
Jd-10a	10212.0	10041.0	8.0	3.0	24 Sept 70	5	3:1	3:1	1.50	-	none
Jd-10b	10207.0	10041.0	8.0	3.0	24 Sept 70	5	3:1	3:1	1.00	-	none
Je-10a	10212.0	10024.0	8.0	3.0	24 Sept 70	5	3:1	3/4:1	40.00	a	none
Je-10b	10207.0	10024.0	8.0	3.0	24 Sept 70	5	3:1	3:1	1.00	-	none

Notes: (1) Coordinates and pier dimensions taken from Bechtel Co. Drawing C-65 Rev 6 dated 5 Jan 1971.
(2) Depths referred to existing rock surface at pier location.
(3) Remark: (a) - Grout communication noted with dewatering system.
(b) - Grout leakage from fissures on vertical pre-split walls.
(c) - Probe water tested below bottom of pier at required grouting pressure. No significant take of water. No proof of grouting done. Probe hole backfilled with 1:1 grout mixture.
(4) Pier marks determined from lettered and numbered construction control lines on Bechtel drawing C-65 Rev 6 dated 5 Jan 1971. Asteris (*) indicates pier line located 6 in. north of construction control line no. 1.

TABLE 2C.5-5

Vane Shear Test Summary

Test No.	Coordinates North ft	Coordinates East ft	Date Made 1970 (1976)	Test Elevation* ft	In-Situ Shear Strength ksf	Remolded Shear Strength ksf	Water Content %	Material Classification
1	10190	11360	30 Oct	576.5	7.0	1.1	18	Till Deposit
2	10190	11360	30 Oct	574.5	3.7	0.9	21	Glaciolacustrine Deposit
3	10190	11360	30 Oct	572.5	3.9	1.3	16	Glaciolacustrine &Till Deposit
4	10150	10850	31 Oct	576.5	7.0	1.1	15	Till Deposit
5	10150	10850	31 Oct	574.5	3.9	1.2	23	Glaciolacustrine Deposit
6	10150	10850	31 Oct	572.5	3.3	1.0	18	Glaciolacustrine &Till Deposit
7	10505	11205	3 Nov	576.5	4.9	1.1	17	Till Deposit
8	10505	11205	3 Nov	574.5	3.0	1.1	26	Glaciolacustrine Deposit
9	10505	11205	3 Nov	572.5	2.0	0.9	29	Glaciolacustrine Deposit
10	10505	11205	3 Nov	571.0	5.5	1.2	26	Glaciolacustrine Deposit
11	10500	10950	4 Nov	576.5	6.8	1.4	15	Till Deposit
12	10500	10950	4 Nov	574.5	4.9	1.4	19	Glaciolacustrine &Till Deposit
13	10500	10950	4 Nov	572.5	3.7	1.4	20	Glaciolacustrine &Till Deposit
14	10150	10610	(8 Jun)	590.0	6.5	0.9	13	Till Deposit

TABLE 2C.5-5 (Continued)

Vane Shear Test Summary

Test No.	Coordinates North ft	Coordinates East ft	Date Made 1970 (1976)	Test Elevation* ft	In-Situ Shear Strength ksf	Remolded Shear Strength ksf	Water Content %	Material Classification
15	10150	10610	(8 Jun)	584.7	8.5	_*	17	Till Deposit
16	10150	10610	(8 Jun)	582.0	6.5	1.6	10	Till Deposit
17	10120	10625	(8 Jun)	587.5	8.5	2.7	15	Till Deposit
10	10120	10625	(8 Jun)	584.7	2.0	1.4	19	Glaciolacustrine Deposit
19	10120	10625	(8 Jun)	582.0	5.7	2.7	21	Glaciolacustrine Deposit

Note

* Elevations referenced to International Great Lakes Datum (IGLD)

* High shear strength — torque wrench broken

2C.6.0 Geotechnical Design Criteria

2C.6.1 Introduction

Geotechnical design criteria were established: for design of foundations for the station structures; for construction of fills to raise station area grades and to form dikes; and for excavation of an Intake Canal. This section presents the design criteria and discusses the factor of safety these criteria represent in terms of the stability of station facilities.

2C.6.2 Foundations

1. General:

Foundations for the station structures consist of mat or strip footings, bearing on bedrock, till deposit, or compacted granular fill and pier footings socketed into bedrock. Specific foundation design characteristics of Class I and major Class II structures are presented in this section.

2. Maximum Design and Ultimate Bearing Capacity for Bearing Materials beneath Mat and Strip Footing Foundations:

Tables 2C.6-1 and 2C.6-2 list the maximum design and ultimate bearing capacities of materials beneath mat and strip footing foundations for station structures. In general, the bearing capacity values are based on the results of laboratory classification and/or strength testing, and consider the results of field explorations. Maximum design values were used to determine footing sizes. Ultimate values were compared to maximum design values in assessing the degree of stability of footing foundations.

Values shown for in-situ glaciolacustrine and till deposit are based on results of laboratory strength tests. Values shown for structural granular fill are based on an assumed 0 value of 40° and a minimum footing width of 5 ft (structural granular fill consists of granular material obtained from the on-site quarry stockpile or approved off-site granular material). Values shown for bedrock are based on results of laboratory strength tests, visual examination of core samples, and in-situ examination of excavations into bedrock.

Table 2C.6-1 lists two values of maximum design bearing capacity for bedrock (50 k/sq ft for treated bedrock and 100 k/sq ft for bedrock free of significant solution activity). A Bedrock Verification Program was implemented in the station area to determine the degree of significant solution activity, if any, in bedrock beneath Class I and major Class II structures. The results of the Bedrock Verification Program indicated that bedrock beneath foundations in the station area was free of significant solution activity and, thus, was considered to have maximum design net bearing pressure of 100 k/sq ft. The ultimate value of bearing capacity shown in Table 2C.6-1 for bedrock free of significant solution activity is based on results of strength tests on rock cores obtained from borings which indicated a minimum with confined compressive strength of 750 t/sq ft and considers the results of borings which indicated the joints in the bedrock to be widely spaced and gypsum filled. On this basis, it is concluded that a value of 600 k/sq ft is a reasonably conservative value for the ultimate bearing capacity of the bedrock free of significant solution activity.

Mat and strip footing foundations were designed using the maximum design values, except when analysis indicated the maximum design values would result in detrimental settlements. In this case, the value used in foundation design was reduced below the maximum design value shown in Table 2C.6-1.

3. Maximum Design and Ultimate Bearing Capacity and Bedrock-Socket Bond Strength for Pier Footing Foundations:

Pier footing foundations support portions of the Auxiliary, Turbine, and Office Buildings. These foundations are socketed into bedrock and are designed to resist anticipated maximum column loads and moments in end-bearing and side shear. The side shear resistance is provided by the bond strength between the bedrock socket and the concrete footing. The following tabulation presents the maximum design bearing capacity and bedrock socket-bond strength values used to determine the diameter and depth of pier socket. This ultimate capacity was compared to the maximum expected loads and moments in assessing the degree of stability of pier foundations.

The ultimate bedrock socket-concrete bond strength shown is based on results on pier load tests made during construction.

Item	Maximum Design Value, k/sq ft	Ultimate Value, k/sq ft
1. Bearing Capacity	100	600
2. Bedrock Socket-Concrete Bond Strength	36	55

4. Foundation for Class I Structures:

a. Containment Building:

The Containment Building is a cylindrical structure 144 ft in diameter[1] consisting of a reinforced concrete shield surrounding a steel Containment Vessel. The Containment Building is founded on a bowl-shaped mat foundation bearing on bedrock free of significant solution activity. The elevation of the bottom of the mat is El. 528 at the center and El. 540 at the perimeter. The maximum height of the Containment Building is 286 ft above El. 528. The minimum thickness of the mat is 4.5 ft. The maximum contact stress beneath the mat under combined static and dynamic loads is 15 k/sq ft. The ultimate bearing capacity of the bedrock is 600 k/sq ft.

b. Auxiliary Building:

The Auxilairy Building is an L-shaped structure that partially surrounds the Containment Building and has three foundation levels.

The northeast portion of the Auxiliary Building is 130 ft by 155 ft in plan and is 70 ft in height above the first floor slab. The first floor is at El. 585 and is supported on grade beams connected to 3-ft-diameter pier footings. Pier footings extend through compacted granular backfill beneath the floor slab and are socketed a minimum of 2.5 ft into bedrock. The maximum load on the pier footings under combined static and dynamic loads is 1700 kips. The ultimate load that can be supported by the bedrock socket, including both end bearing and side shear load resistances, is approximately 4900 kips.

[1] Dimensions given for station area structures are approximate.

The southeast portion of the Auxiliary Building is 130 ft by 140 ft in plan and is 120 ft in height above the foundation level. It is supported on a 3-ft-thick mat foundation that bears on bedrock at El. 542. The maximum contact stress beneath the mat under combined static and dynamic loads is 7 k/sq ft. The ultimate bearing capacity of the bedrock is 600 k/sq ft.

The southwest portion of the Auxiliary Building is 140 ft by 140 ft in plan and is 120 ft in height above foundation level. It is supported on a 2-ft-thick mat foundation; the outside walls are supported on 3-ft-thick strip footings. The bottom of the mat is at El. 563 and is underlain by an approximately 2-ft-thick layer of concrete backfill over bedrock. For these conditions, the mat can be considered to be supported on bedrock. The bottoms of the strip footings bear on bedrock at El. 557. The maximum contact stress beneath the mat and strip footings under combined static and dynamic loads is 7 k/sq ft. The ultimate bearing capacity of bedrock is 600 k/sq ft.

c. Intake Structure:

The Intake Structure is 58 ft by 62 ft in plan and is 55 ft in height above foundation level. The Intake Structure is founded on a 3-ft-thick mat foundation bearing on bedrock at El. 543. The maximum contact stress beneath the mat under combined static and dynamic loads is 26 k/sq ft. The ultimate bearing capacity of the bedrock is 600 k/sq ft.

d. Service Water Pipe Tunnel:

The Service Water Pipe Tunnel is a 12-ft-high, 11-ft-wide, and 360-ft-long reinforced concrete tunnel between Service Water Valve Room No. 1 in the Turbine Building and Service Water Valve Room No. 2 adjacent to the Intake Structures. The walls and roof of the tunnel are 2-ft-thick and are supported on a 2-ft-thick mat foundation. The roof of the tunnel is covered by approximately 7 ft of compacted granular fill. The bottom of the mat foundation is at El. 564.5 and bears on Class I compacted granular fill or on till deposit. The maximum contact stress beneath the mat under combined static and dynamic loads is 4 k/sq ft. The ultimate bearing capacities of the till deposit and the Class I compacted granular fill are 40 k/sq ft and 50 k/sq ft, respectively.

e. Service Water Valve Rooms No. 1 and 2:

Service Water Valve Rooms No. 1 and 2 are located on opposite ends of, and are contiguous to, the Service Water Pipe Tunnel.

Service Water Valve Room No. 1 is approximately 39 ft by 33 ft in plan and is 20 ft in height above the floor slab elevation. It consists of 3-ft-thick reinforced concrete walls and roof supported on a 3.5-ft-thick mat foundation. The bottom of the mat bears on bedrock at El. 561.5.

Service Water Valve Room No. 2 is approximately 26 ft by 58 ft in plan and is 19.5 ft in height above the floor slab elevation. It consists of 2-ft-thick reinforced concrete walls and roof supported on a 2-ft-thick mat foundation. The bottom of the mat is at El. 564 and bears on approximately 6 ft of concrete backfill above bedrock. For these conditions, Service Water Valve Room No. 2 is considered to be founded on bedrock. The maximum contact stress beneath the mat for both buildings under combined static and dynamic loads is 4 k/sq ft. The ultimate bearing capacity of bedrock is 600 k/sq ft.

f. Borate Water Storage Tank:

The Borate Water Storage Tank is 47 ft in dia and is 50 ft in height above foundation level. It is located west of the Containment Building. The tank consists of a 1/4-in-thick steel shell and roof. The tank shell is supported on a reinforced concrete mat foundation bearing on Class I granular backfill at El. 585. The Class I backfill extends from El. 585 to the top of bedrock at approximately El. 560. The maximum contact stress beneath the mat under combined static and dynamic loads is 7.5 k/sq ft. The ultimate bearing capacity of the compacted granular fill is 50 k/sq ft.

g. Electrical Manholes No. 3001, 3004, 3005, 3006, 3020, 3041, and 3042:

Electrical Manholes No. 3001, 3004, 3005, 3006, 3020, 3041, and 3042 are supported on mat foundations. The mat foundations bear on Class I compacted granular backfill. The maximum contact stress beneath the mat under combined static and dynamic loads is 2 k/sq ft. The ultimate bearing capacity of the Class I compacted granular fill is 50 k/sq ft.

5. Foundations for Major Class II Buildings:

a. Turbine and Office Buildings:

The Turbine and Office Buildings are approximately 170 ft by 270 ft in plan. The Turbine Building is 110 ft in height above first floor level (El. 585); the Office Building is 75 ft in height above first floor level. Building loads are supported on pier footings with diameter or maximum rectangular dimension ranging from 3.0 ft to 14.0 ft. Piers are socketed from 3.0 ft to 8.0 ft into bedrock. The piers are designed to resist maximum axial loads and moments resulting from combined static and dynamic loads.

The ratio between the ultimate bedrock socket load capacity and maximum expected pier load was calculated for each pier footing assuming an ultimate bearing capacity for bedrock of 600 k/sq ft and an ultimate bedrock socket concrete bond strength of 55 k/sq ft. The ratio ranged from 2.6 to 8.7.

b. Turbine-Generator Foundation:

The Turbine-Generator is supported on a reinforced concrete pedestal with columns integrally connected to a 9-ft-thick mat foundation. The mat foundation has plan dimensions of 44 ft by 175 ft and bears on bedrock at El. 558. The maximum contact stress beneath the mat under combined static and dynamic loads is 12.5 k/sq ft. The ultimate bearing capacity of the bedrock is 600 k/sq ft.

c. Water Treatment Building:

The Water Treatment Building is 93 ft by 95 ft in plan and is 60 ft in height above foundation bearing on bedrock at El. 558. The maximum contact stress beneath the foundation under combined static and dynamic loads is 25 k/sq ft. The ultimate bearing capacity of the bedrock is 600 k/sq ft.

d. Cooling Tower:

The Cooling Tower is located 1350 ft northwest of the Containment Building. The Cooling Tower consists of a reinforced concrete shell 415 ft in diameter at its base and 493 ft in height above the basin sill. The Cooling Tower shell is supported on a 13-ft-wide ring footing which bears on 6 ft to 8 ft of compacted granular fill at approximately El. 572. The compacted granular fill is underlain by 4 ft to 6 ft of till deposit which is underlain by bedrock. The maximum contact stress beneath the ring footing under combined static and dynamic loads is 9.5 k/sq ft. The ultimate bearing capacity of the granular fill is 50 k/sq ft.

The Cooling Tower fillers are attached to columns supported on strip footings. The strip footings are monolithic with the cooling basin floor slab and bear on a 3-ft-thick layer of compacted granular backfill at El. 575. The compacted granular backfill is underlain by in-situ glaciolacustrine deposit. The maximum contact stress beneath the strip footings under combined Static and dynamic loads are 4 k/sq ft. The ultimate bearing capacity of the in-situ glaciolacustrine deposit is 10 k/sq ft.

The cooling tower basin has been remedially grouted to prevent excessive settlements.

6. Settlements of Class I and Major Class II Structures:

The settlements of Class I and major Class II structures were estimated assuming that materials beneath the foundations for these structures (bedrock, till deposit, and structural granular fill material) were linearly elastic within the stress ranges for which settlements were estimated. The maximum contact stresses and distribution of materials beneath foundations used in the settlement estimate for particular structures are those discussed in Subsection 2C.6.2(4) and particular structures are those discussed in Subsections 2C,6.2(4) and 2C.6.2(5) of this section. The value of Young's modulus used in the settlement analyses for each material is as follows:

Material	Young's Modulus, E. k/sq ft
1. Bedrock	350×10^3
2. Till Deposit	1.0×10^3
3. Structural Granular Fill	1.5×10^3

The value of Young's modulus for bedrock and till dcposit is based on results of static laboratory compression tests. The value of Young's modulus for structural granular fill is based on results of plate load tests made in the Cooling Tower and station area.

Based on these settlement analyses, it is estimated that maximum settlements of Class I and major Class II structures founded on bedrock (i.e., Containment Building, Auxiliary Building, Turbine and Office Buildings, Intake Structure, and Valve Room No. 1) will be less than 1/8 in. and that settlements of Class I structures founded on till deposit and granular fill (Borate Water Storage Tank, Pipe Tunnel, and Valve Room No. 2) will be less than 1/4 in. It is estimated that settiments of the major Class II structures founded on significant depths of till deposit or granular fill material (Cooling Tower shell and basin) will be less than 3/4 in.

2C.6.3 Fills

1. General:

Fill construction at the site consisted of the following: (1) non-Class I general fill in the station area; (2) Class I Intake Forebay Dike fill; (3) non-Class I Wave Protection Dike fill; (4) non-Class I granular backfill in the Cooling Tower area; and (5) Class I and non-Class I granular backfills in the station area. The following text presents the geotechnical design criteria for each type of fill.

2. Non-Class I General Fill in the Station Area:

Approximately 808,000 cu yd of non-Class I general fill (hereafter referred to as general fill) was placed and compacted in the vicinity of the station area to raise grades from approximately El. 574 to El. 583 and for construction of an Intake Canal beyond station 7+00 (see Bechtel Drawing for location of station area fills and intake canal). General fills consisted of compacted glaciolacustrine and till deposits obtained from four on-site borrow areas.

General fill was to be compacted in lifts not exceeding 12-in. in thickness with a minimum of 6 coverages of an approved sheepsfoot roller. General fill material was to be placed and compacted within a water content range of in-situ water content to optimum water content minus 5 percentage points. No Class I or major Class II structures are founded on general fill material.

General fill used during the operational phase of the plant are placed in accordance with the design drawings and specifications governing the modification activity.

3. Class I Intake Forebay Dike Fill:

Approximately 100,000 cu yd of Class I Intake Forebay Dike fill (hereafter referred to as Class I intake fill) was placed and compacted along the Intake Canal to El. 582 between approximately station 0+00 and approximately station 7+00; Figure 2C.6-1 shows the location of Class I intake fill. Placement of fill occurred during the period May through October 1970 and August through October 1972. Class I intake fill material consists of compacted glaciolacustrine and till deposit obtained from on-site borrow areas.

Class I intake fill was to be compacted in 12-in.-thick lifts with a minimum of 4 coverages of an approved sheepsfoot roller. Class I intake fill material was to be placed and compacted within a water content range of in-situ water content to optimum water content minus 5 percentage points.

4. Non-Class I Wave Protection Dike Fill:

Approximately 91,000 cu yd of non-Class I Wave Protection Dike fill (hereafter referred to as Wave Protection Dike fill) was placed and compacted to El. 591 along the north, east, and a small portion along the south sides of the station area (see Figure 2C.6-1). Wave Protection Dike fill material consisted of topsoil obtained from the on-site topsoil stockpile.

Wave Protection Dike fill was to be placed in 12-in.-thick lifts within a water content range of 20 to 37 percent and compacted with a minimum of 6 coverages of an approved sheepsfoot roller.

5. Non-Class I Granular Backfill in the Cooling Tower Area:

Approximately 47,000 cu yd of non-Class I granular backfill was placed and compacted in the Cooling Tower area. The backfill was placed and compacted as bearing material for the ring footing supporting the Cooling Tower shell, and as bearing material for the Cooling Tower basin. The granular backfill material consisted of crushed rock obtained from the on-site quarry stockpile.

Granular fill material beneath the ring footing was to be placed in loose lifts not exceeding 6 in. in thickness and compacted using vibratory compaction equipment to 80 percent of relative density determined in accordance with ASTM Specification No. D-2049. Granular backfill material beneath the Cooling Tower basin was to be placed in loose lifts not exceeding 6 in. in thickness and compacted using vibratory compaction equipment to 70 percent relative density determined in accordance with ASTM Specification No. D-2049.

Granular fill used in the Cooling Tower area during the operational phase of the plant are placed in accordance with design drawings and specifications goveming the modification activity.

6. Granular Backfill in the Station Area:

Approximately 110,000 cu yd of granular backfill was placed and compacted in the station area as shown in Figure 2C.6-3.

Granular backfill material consisted of crushed rock obtained from the on-site quarry stockpile with approximate gradation characteristics as those required in the project specifications.

Class I and non-Class I structural backfill (hereafter referred to as structural backfill) were to be placed in loose lift thicknesses ranging from 12 in. in large work areas[1] to 6 in. in small work areas[2] . Structural backfill was to be compacted to 98 percent of the maximum dry density determined in accordance with ASTM Specification No. D698 Method D, latest revision or to 80 percent relative density determined in accordance with ASTM Specification No. D2049. General backfill was to be placed in loose lift thicknesses ranging from 14 in. in large work areas to 7 in. in small work areas. General backfill was to be compacted to 95 percent of the maximum dry density determined in accordance with ASTM Specification No. D698 Method D, latest revision. Water content of granular fill material was to be no less than 3 percent nor more than 12 percent determined on a dry weight basis.

7. Maximum Design and Ultimate Bearing Capacity of In-Situ Fill:

Table 2C.6-2 lists the maximum design and ultimate bearing capacity for all classifications of in-situ compacted fill. For cohesive fill materials (till deposit and glaciolacustrine deposit), the values shown are based on the compactive effort specified in the pertinent project specifications, on results of laboratory undrained strength testing, and on results of observations and laboratory tests made during placement and compaction of fill material. For cohesionless

---

[1] Large work areas are defined as work areas where compaction is accomplished with a minimum of 4 coverages of a Raygo 400, or approved equivalent compactor.

[2] Small work areas are defined as work areas where compaction is accomplished with a minimum of 4 coverages of a Super Ground Pounder Model 7000, or approved equivalent compactor.

fill material (granular material from on-site quarry), the values shown are based on compactive effort specified in the pertinent project specifications, on results of plate load tests made in the station area and the Cooling Tower area, and on observations of placement and compaction of the material in the station area. The values shown assume a minimum value of f of 40° for granular material compacted to 80 percent relative density and a minimum value of f of 35° for granular material compacted to 70 percent relative density. In addition, the values shown assume a minimum footing width B of 5 ft.

Where necessary, the bearing capacity used in design of foundations was reduced below the maximum design values to limit total or differential settlements.

2C.6.4 Intake Canal

1. General:

A 2800-ft-long Intake Canal was constructed in a northeast direction from the Intake Structure to the lake shoreline. The invert elevation of the canal ranges from approximately El. 546 at the intake structures to El. 558 near the lake. The Intake Canal does not extend to the lake. A 30-ft-thick soil dike remains between the end of the canal and the lake and a 96-in.-dia pipe extends through the soil dike to a point approximately 3300 ft offshore. Intake water required for plant operation flows into the Intake Canal through this pipe.

The width of the Intake Canal, measured between the dike crest centerlines, ranges from 430 ft between station 0+00 to approximately 7+00 (Class I portion) to 270 ft beyond station approximately 7+00 (non-Class I portion). The dike slopes of outboard canal are 3:1 (3 horizontal to 1 vertical) from Station 0+00 to approximately Station 10+00. The inboard side of the canal varies with a nominal slope of 2.5:1 with a few localized areas down to 2.14:1 from station 0+00 to approximately station 10+00. From approximately Station 10+00 to Station 27+50, the inboard side of the canal varies with a slope of 3:1 and a slope of 2:1 with a few localized areas down to a 1.5:1 slope. The invert of the canal is in till deposit except between station 0+00 and approximately station 2+00 where the invert is in bedrock. The inboard side of the 3:1 canal slopes are lined with a 3-ft-thick facing of random placed angular quarry stone between station 0+00 and station 5+50. Beyond station 5+50, the canal dike slopes are also lined with smaller size riprap with a depth generally less than 3 feet thick. The canal invert and outboard side of the canal dike are unlined.

2. Stability of Class I Canal Dikes:

The extended duration condition is associated with the dike slope stability after a period of time during which the groundwater levels (pore-water pressures) within the dike reach an equilibrium condition with the water level inside the canal and the surrounding land around the dike. The length of this period may vary from a few months for relatively permeable soils to several years for low-permeability soils. This is the condition addressed in the Reference 59 evaluation since the dikes were constructed approximately three decades ago.

A stability analysis was made to determine the factor of safety of the Class I Intake Canal Dikes during occurrence of a Maximum Possible Earthquake (larger earthquake). The factor of safety is defined as the ratio of the available undrained shear strength along the same failure surface required to provide a factor of safety of 1.0 for assumed loading conditions.

For purposes of the analyses, the water level in the Intake Area was assumed to be at El. 576 and a horizontal force equal to 0.15 times the weight of the sliding mass was assumed to act in the direction of sliding as documented in Reference 59.

Factors of safety were calculated for several assumed failure surfaces which would result in the lowest factor of safety. Circular arc and sliding wedge modes of failure were analyzed to determine the required undrained shear strength along the failure surface for the assumed loading conditions. The sliding wedge mode of failure resulted in the minimum factor of safety. As documented in the Reference 59 evaluation, the sliding wedge failure remains the appropriate failure mode as dictated by the soil properties. Figure 2C.6-2 shows the dike cross section, material distribution, and water level for the seismic loading conditions. Results of the Reference 59 evaluation, indicate that the slope stability of the dike for the sliding wedge mode of failure for the current (as-built) conditions is acceptable.

The undrained shear strength characteristics of the compacted glaciolacustrine and till deposits were determined in two series of strength tests. Prior to construction, a series of Unconsolidated-Undrained (UU) triaxial tests were made on samples of compacted glaciolacustrine and till deposits (compaction was in accordance with ASTM Specification No. D698-68T Method A). The UU tests indicated that the undrained shear strength of compacted glaciolacustrine deposit ranged from approximatley 2.6 k/sq ft as a water content of 20 percent to 1.0 k/sq ft as a water content of 25 percent; and that the undrained shear strength of compacted till deposit ranged from 5.0 k/sq ft at a water content of 13 percent to 0.6 k/sq ft as a water content of 19 percent. It is concluded, on the basis of test procedures used to make the UU tests and on the basis of the plasticity indices of the material tested, that the data obtained represent the lower bound of undrained shear strength of compacted glaciolacustrine and till deposits. Reference 59 indicates that the soil parameters used to evaluate soil stability are consistent with this range of parameters.

During construction, a series of thirteen in-situ vane shear strength tests was made in the Class I fill at the four locations shown in Figure 2C.6-1. A discussion of these tests is presented in Section V of this Appendix. These tests indicated that the vane shear strength of compacted glaciolacustrine deposits range from approximately 4.0 k/sq ft at a water content of 20 percent to 2.0 k/sq ft at a water content of 29 percent; and that the vane shear strength of compacted till deposit ranged from 7.0 k/sq ft at a water content of 15 percent to 3.5 k/sq ft at a water content of 19 percent. It is concluded, on the basis of test procedures used that the data obtained from the vane tests represent the upper bound of undrained shear strength of compacted glaciolacustrine and till deposits. Reference 59 indicates that the soil parameters used to evaluate soil stability are consistent with this range of parameters.

Water content tests were made on samples of the in-place fill obtained during inspection of placement and compaction of Class I fill.

Ninety-seven (97) water content tests made on till deposit samples indicated the average water content of till deposit placed and compacted as Class I fill was approximately 16 percent. One hundred forty-four (144) water content tests made on glaciolacustrine deposit samples indicated the average water content of glaciolacustrine deposits placed and compacted as Class I fill was approximately 25 percent.

Reference 59 Evaluates the sliding wedge mode of failure and indicates a minimum factor of safety of 1.30 for the Class I dikes during application of Maximum Possible (larger) Earthquake forces.

TABLE 2C.6-1

Summary of Maximum Design and Ultimate Bearing Capacities for Soil and Bedrock

	Bearing Material	Maximum Design Bearing Capacity k/sq ft	Ultimate Bearing Capacity k/sq ft
1.	In-situ glaciolacustrine deposit	4	10
2.	In-situ till deposit	10	40
3.	Bedrock found to contain significant solution activity (after satisfactory completion of remedial treatment procedures	50	250
4.	Bedrock free of significant solution activity	100	600
5.	Compacted fill (see Table VI-2)	--	--

TABLE 2C.6-2
Summary of Maximum Design and Ultimate Bearing Capacities for Compacted Fill Materials

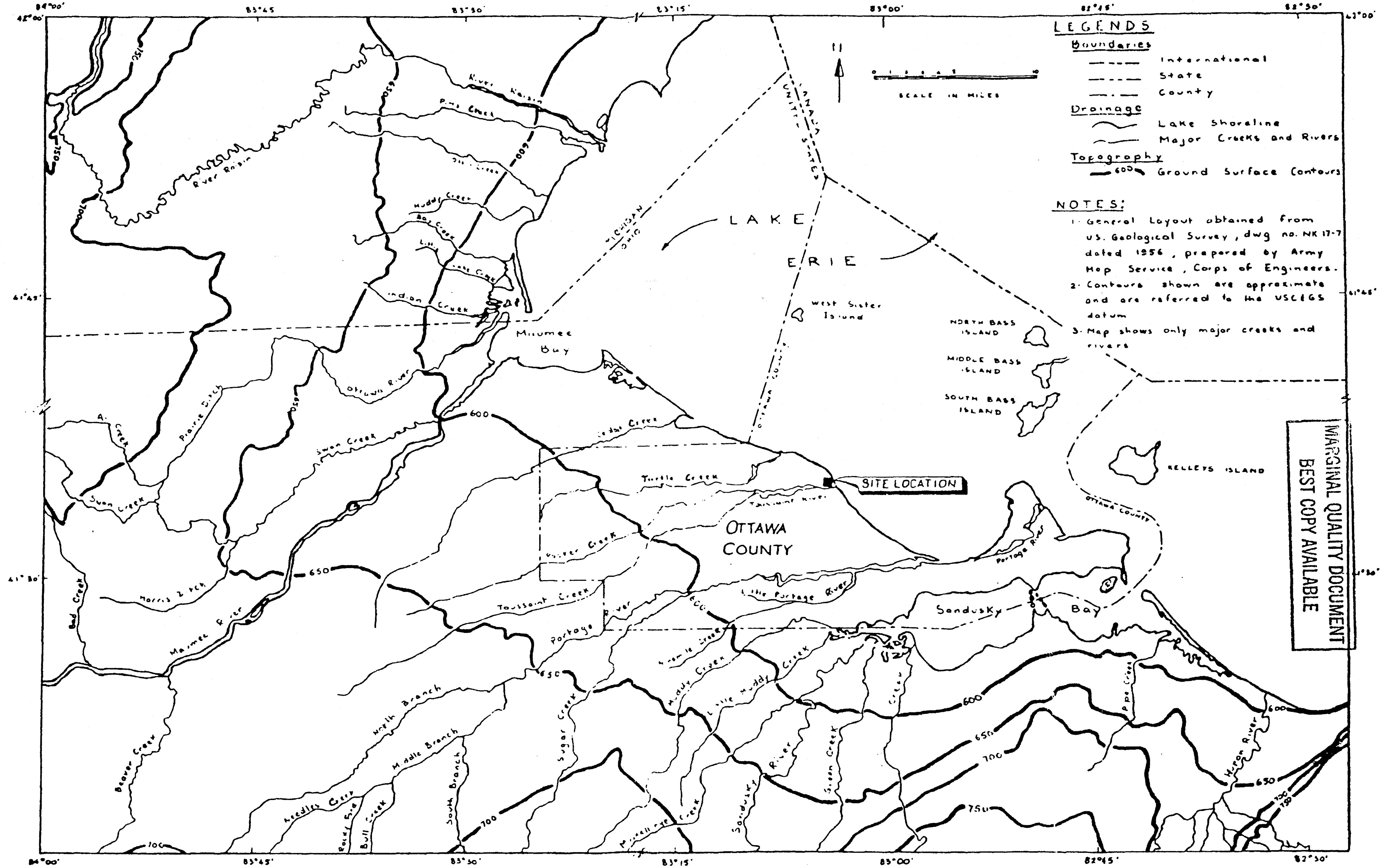
Fill Material	Maximum Design[2] Bearing Capacity k/sq ft	Ultimate Bearing[2] Capacity k/sq ft
1. Non-Class I general fill in the station area	3[1]	15
2. Class I intake forebay dike fill	1.5[1]	7
3. Non-Class I wave protection dike fill	N.A.	–
4. Non-Class I granular backfill in the cooling tower area [3]		
a. Compacted to 80 percent relative density	10	50
b. Compacted to 70 percent relative density	4[1]	20
5. Granular backfill in the station area [3]		
a. Class I and non-Class I structural backfill	10	50
b. Non-Class I general backfill	4[1]	20

---

1 No Class I or major Class II structures founded on this classification of fill.

2 Values are reduced as necessary where compacted fill is underlain by more compressible materials.

3 Values given are for the construction phase of the plant. Bearings values for the operational phase can be found in the Design Specifications, as applicable.



PHYSIOGRAPHIC MAP OF SITE REGION

WMAI 68-192

FIGURE 2C.2-1
REVISION 0
JULY 1982

PHOTO TAKEN 31 JULY 1971

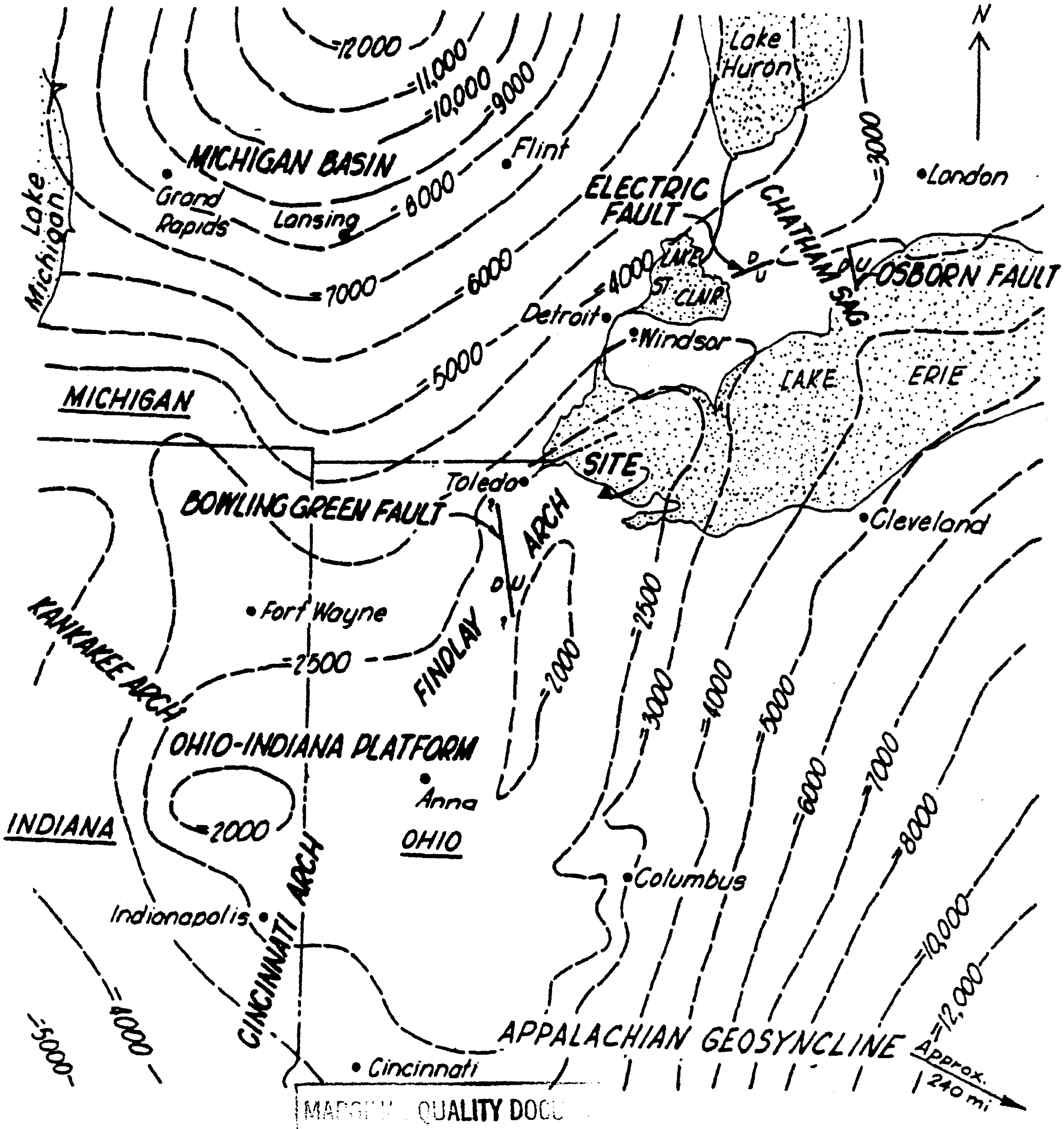
0 1000 2000 FT
SCALE (APPROX)

AERIAL ~~PHOTOGRAPH~~ SHOWING
PHYSIOGRAPHIC FEATURES OF THE SITE AREA

WMAI 68-192

FIGURE 2C.2-2

REVISION 0
JULY 1982



-5000— Elevation of Precambrian bedrock surface
(Referenced to USC & GS Datum)

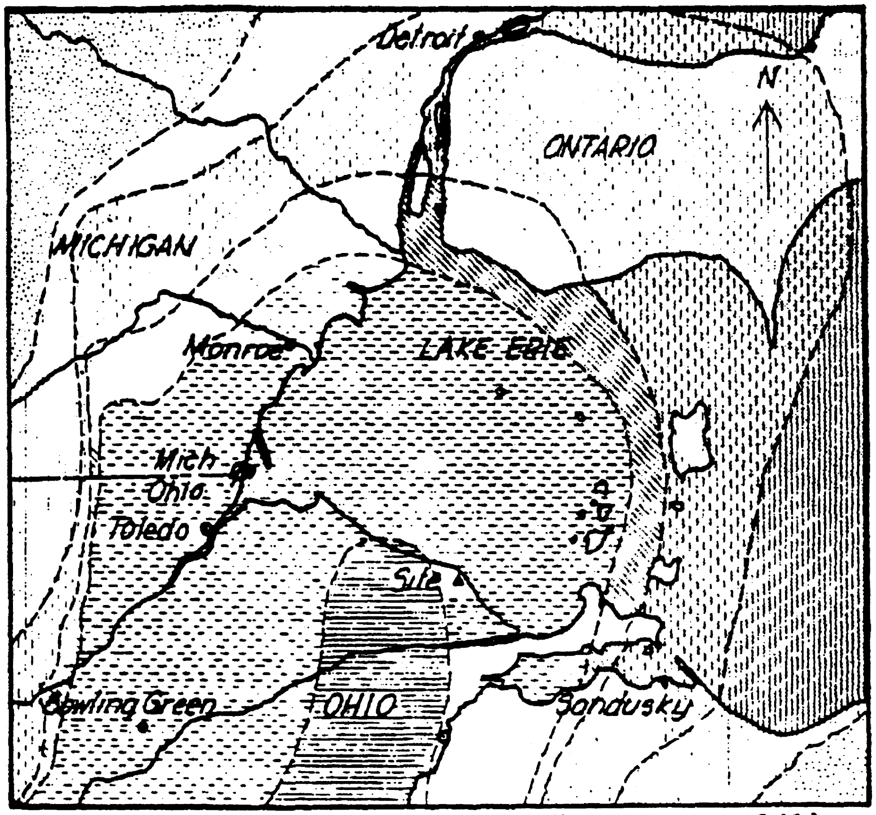
0 40mi
Scale

(Base map: modified after Basement Map of North America, USGS and AAPG, 1967)

REGIONAL CONTOURS OF PRECAMBRIAN SURFACE AND PRINCIPAL STRUCTURAL FEATURES

WMAI 68-192

FIGURE 2C.2-3
REVISION 0
JULY 1982



(Modified after Carman, 1946)

0 16mi

Scale

Legend

- Miss: Waverly formation
- Dev: Ohio formation
- Dev: Columbus formation
- Dev: Detroit River group
- Silur: Bass Island group
- Silur: Niagara group

REGIONAL BEDROCK MAP

FIGURE 2C.2-4

REVISION 0

JULY 1982

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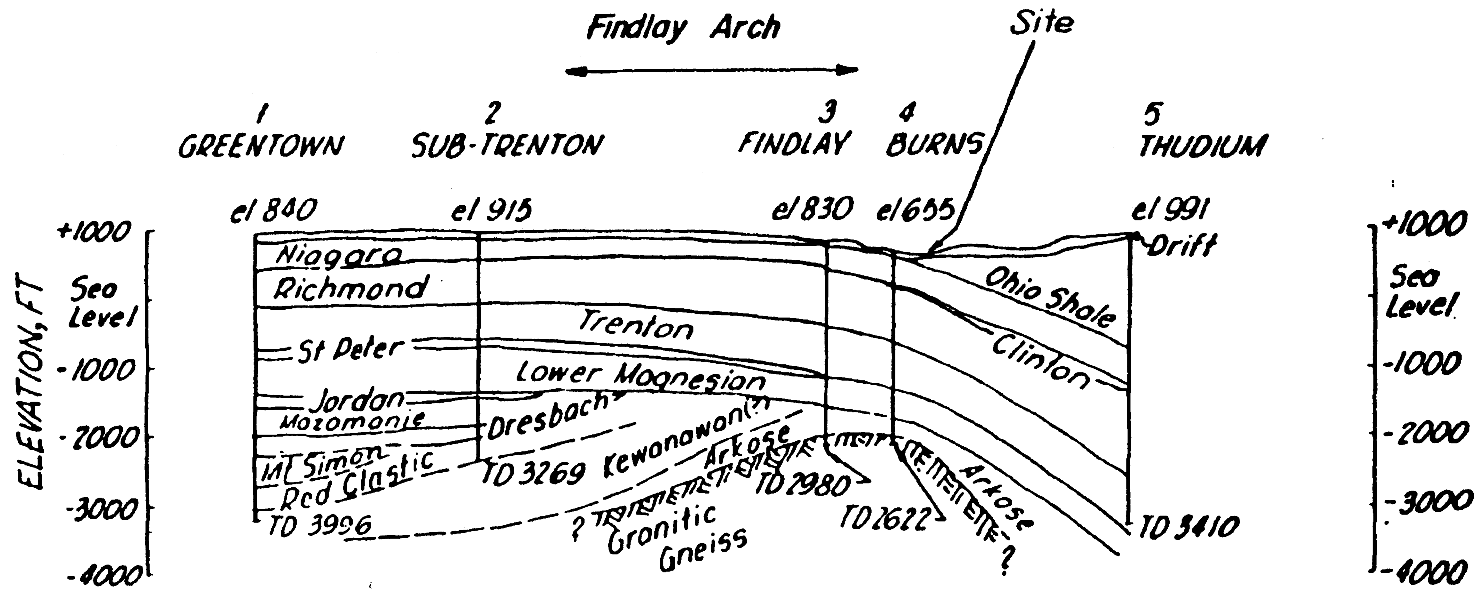
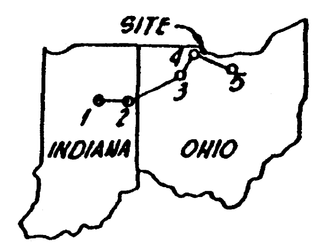
REGIONAL CROSS SECTION



NOTES

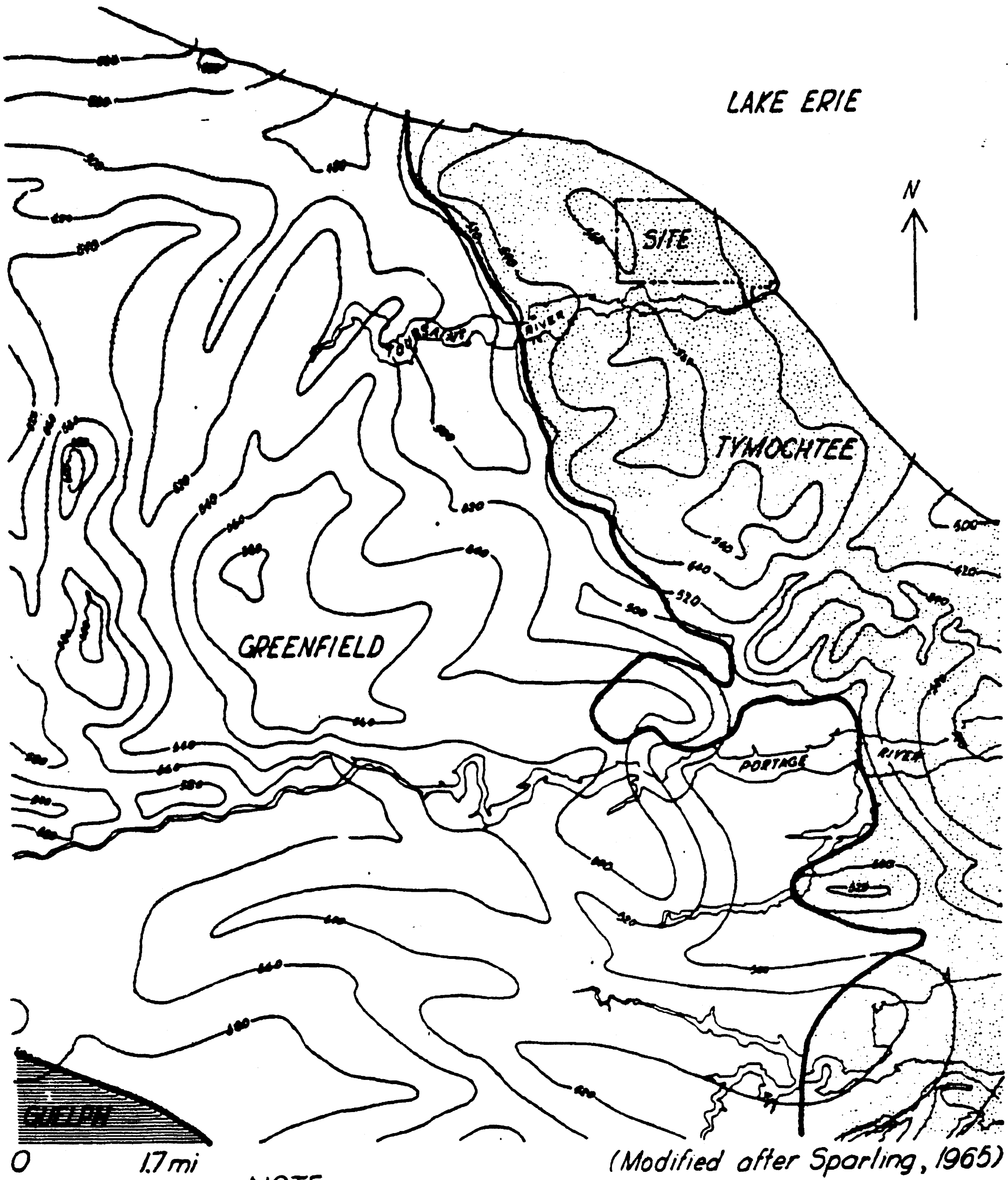
1. Distortion 87(hor.): 1(vert)
2. Elevation referenced to USC & GS Datum

0 50 mi
Horizontal scale



(Modified after Wasson, 1932)

MARGINAL QUALITY DOCUMENT
BEST COPY AVAILABLE



NOTE
Elevations referenced to USC&GS Datum

INFERRED LOCAL BEDROCK SURFACE CONTOURS AND GEOLOGIC MAP

MARGINAL QUALITY DOCUMENT
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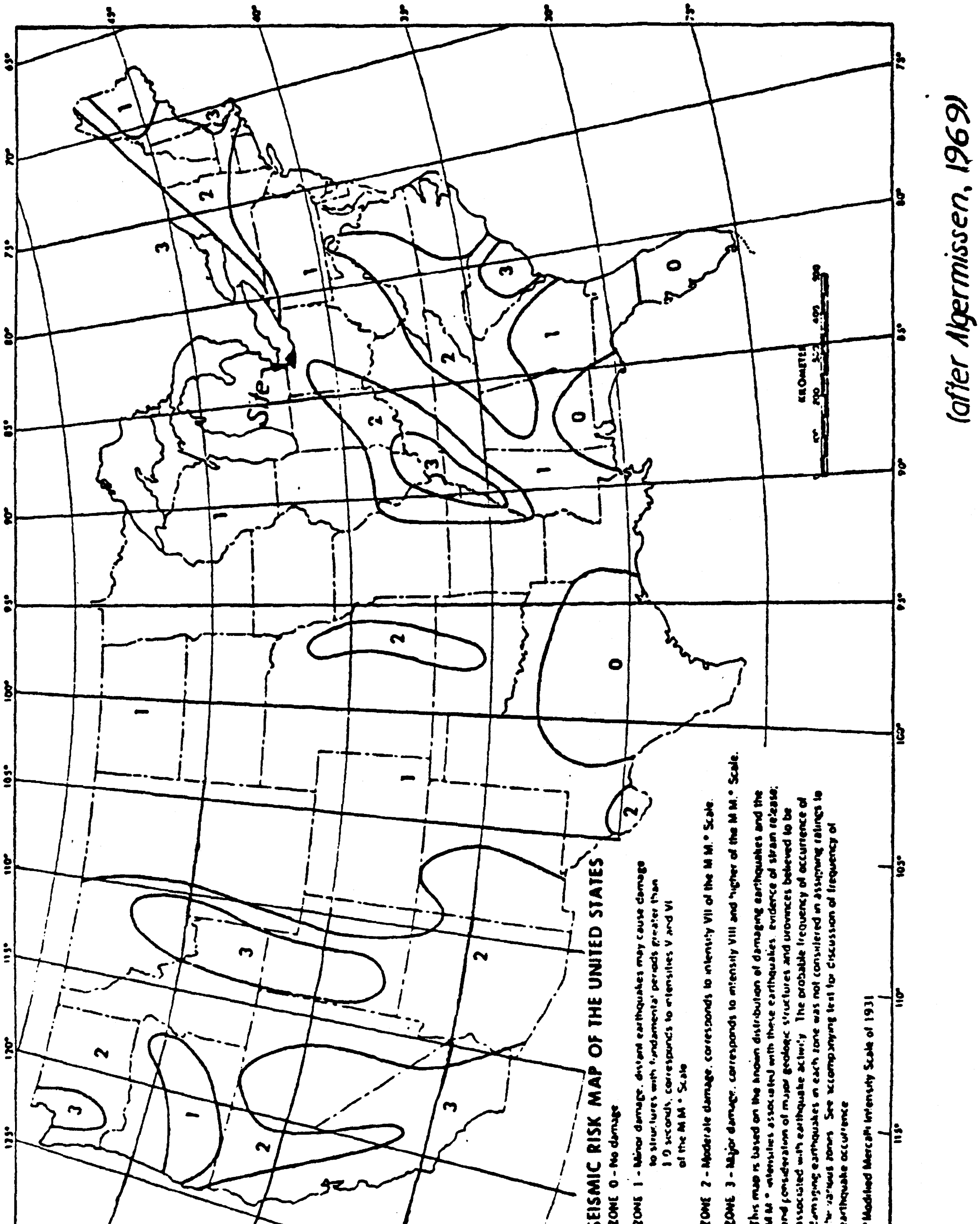
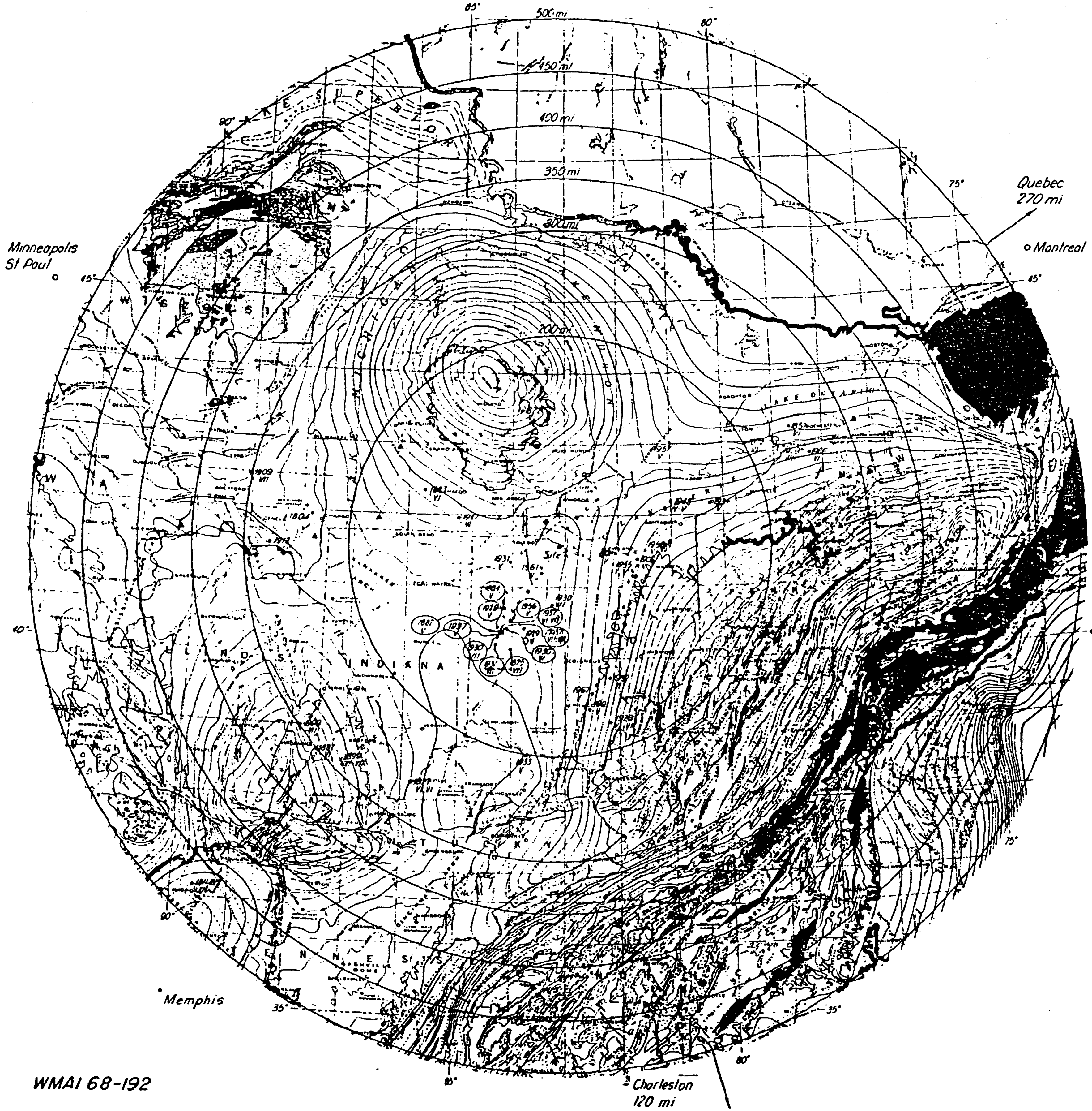
WMAI 68-192

FIGURE 2C.2-6
REVISION 0
JULY 1982



SEISMIC-RISK MAP OF THE U.S.A.



WMAI 68-192

LEGEND

•	Earthquake epicenter
1947	Year of occurrence
VI	Modified Mercalli epicentral intensity
▲	Nuclear power station location
0-200mi	Earthquakes ≥ MM V
200-300mi	" ≥ MM VI
300-350mi	" ≥ MM VII
350-400mi	• ≥ MM VIII
400-450mi	• ≥ MM IX
450-500mi	• ≥ MM X

Contour lines are those of significant bedrock formations; i.e., Trenton limestone, Oriskany sandstone, and Precambrian basement rock, et are below USC & GS datum

REFERENCES

USC&GS Earthquake History of the United States, 1965

USC&GS United States Earthquakes (individual years 1929 through 1970)

Dominion Observatory of Canada, 1960, 1961, 1962(?), 1963-1966

Westland et al, 1940

Bradley et al, 1965

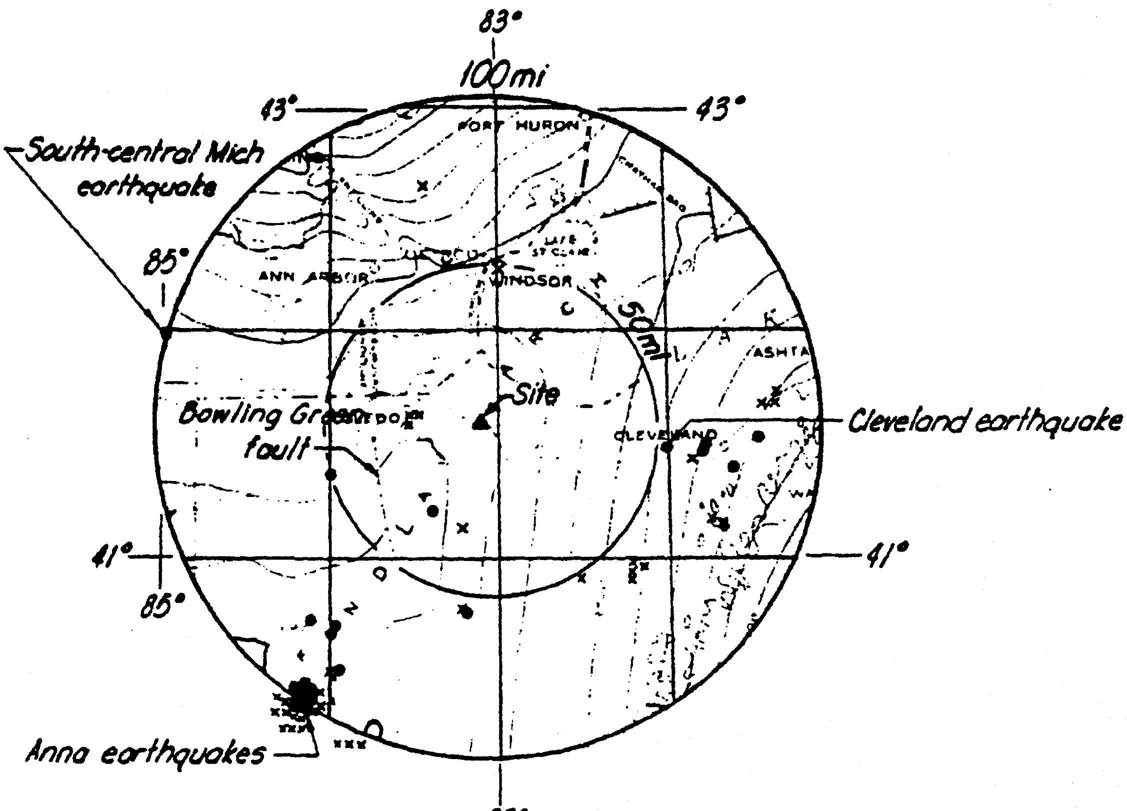
Bulletin of the Seismological Society of America, February 1969 - June 1972 listing earthquakes May 1969 - October 1971

Base map: "Tectonic Map of the United States" USGS and AAPG, 1962

0 50 100 mi
scale

EARTHQUAKE EPICENTER MAP

FIGURE 2C.3-2
REVISION 0
JULY 1982



LEGEND

● Earthquakes ≥ MM V
× Earthquakes < MM V

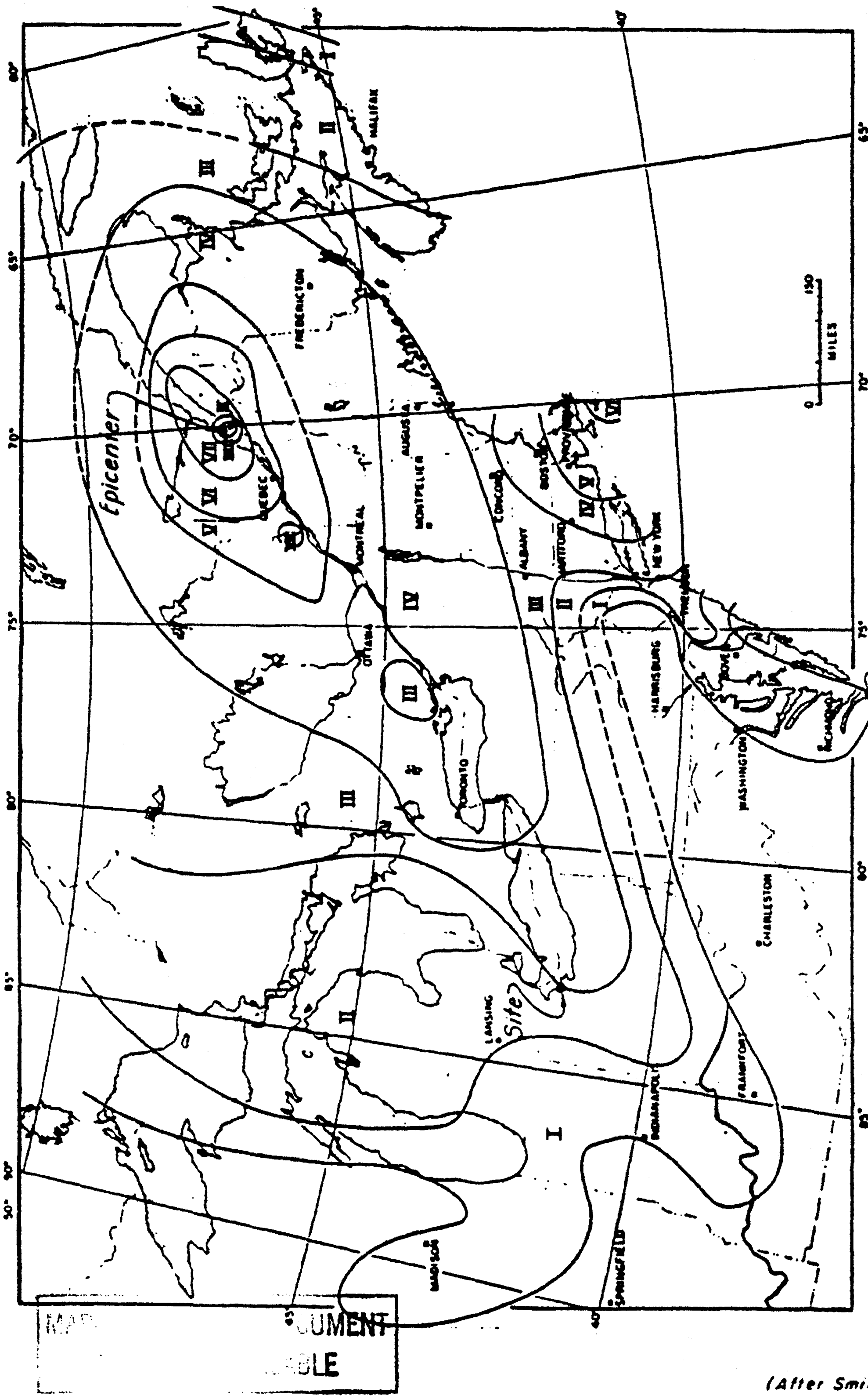
0 50mi
Scale

(Base map after USGS and AAPG 1962)

EARTHQUAKE EPICENTERS WITHIN APPROX 100 MI FROM SITE

WMAI 68-192

FIGURE 2C.3-3
REVISION 0
JULY 1982

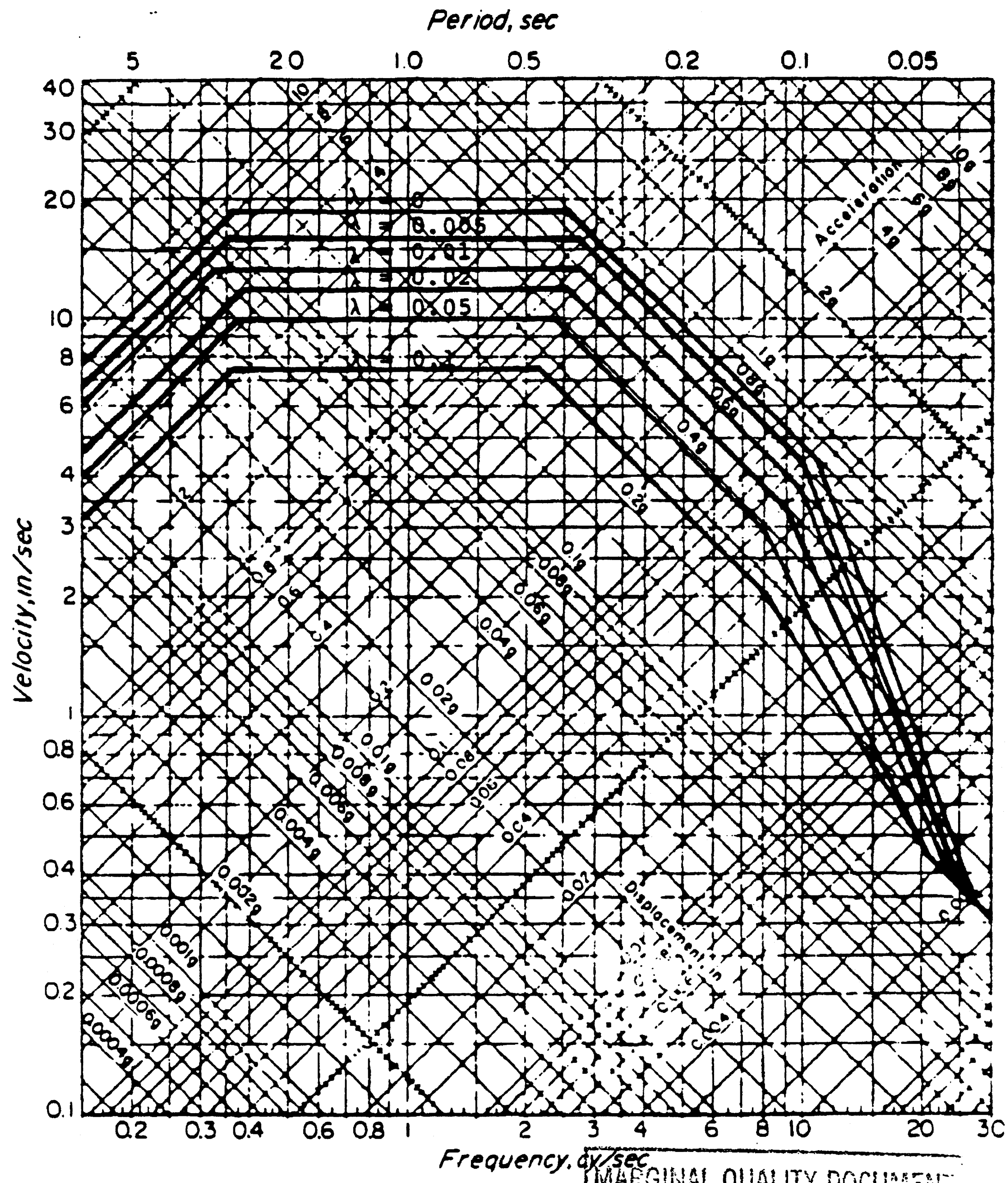


(After Smith, 1966)

ISOSEISMAL MAP OF ST LAWRENCE EARTHQUAKE OF 1 MAR 1925

WMAI 68-192

FIGURE 2C.3-4
REVISION 0
JULY 1982



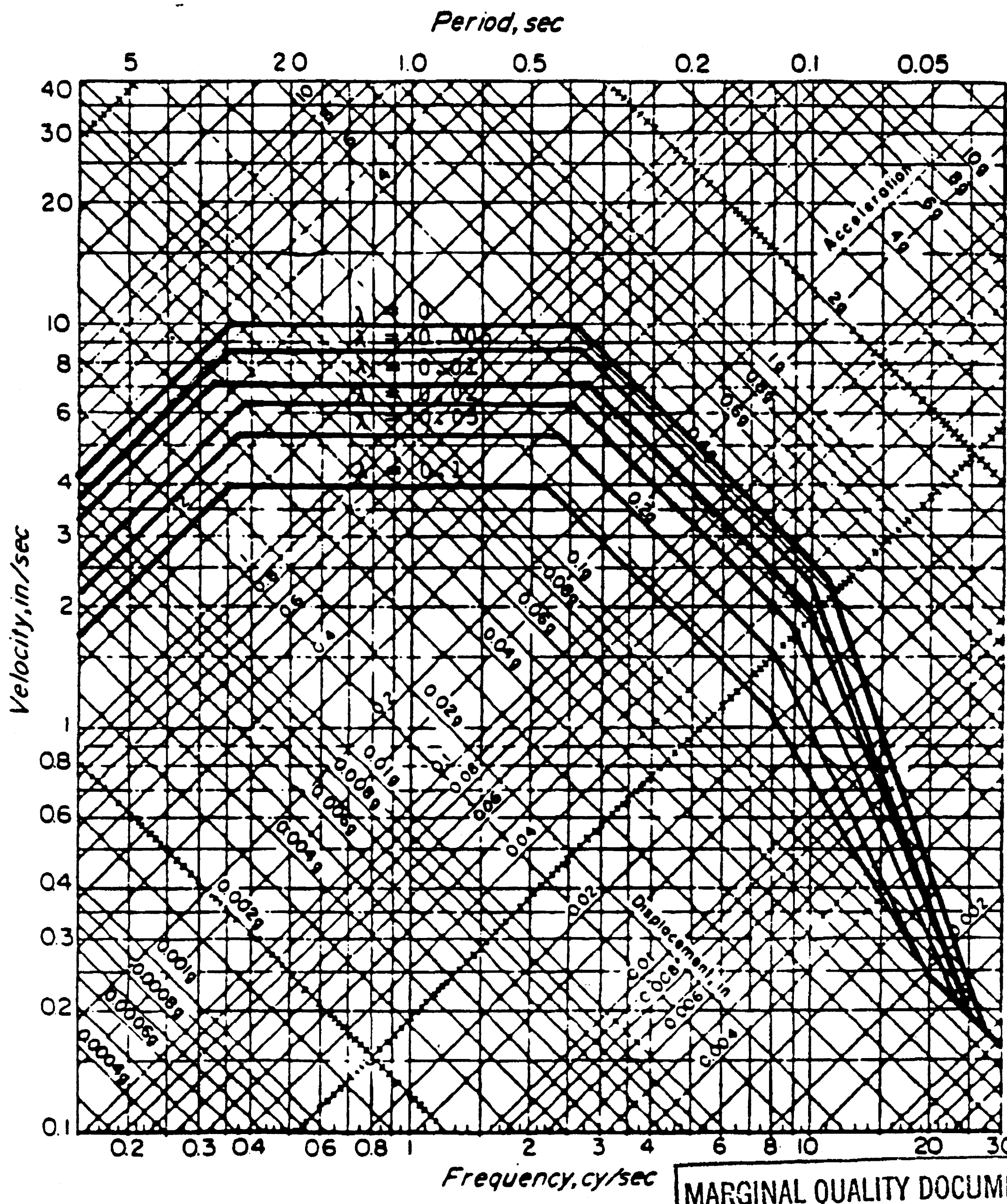
λ = Ratio of critical damping

MARGINAL QUALITY DOCUMENT
BEST COPY AVAILABLE

RECOMMENDED RESPONSE SPECTRA FOR HORIZONTAL
VIBRATORY GROUND MOTIONS OF
MAXIMUM POSSIBLE EARTHQUAKE (LARGER EARTHQUAKE)
(0.15 g)
FOR SEVERAL DAMPING RATIOS

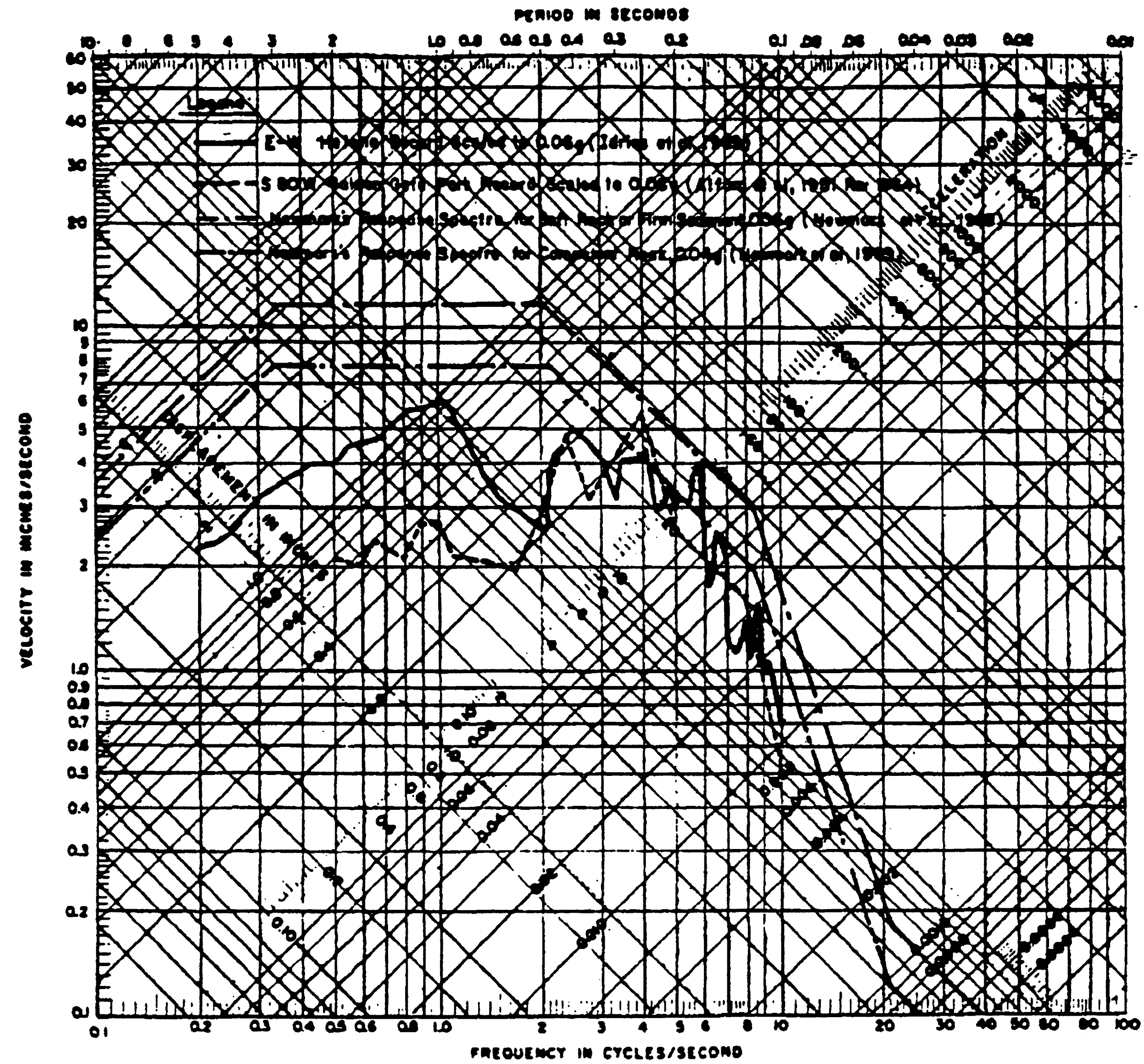
WMAI 68-192

FIGURE 2C.3-5
REVISION 0
JULY 1982



λ = Ratio of critical damping

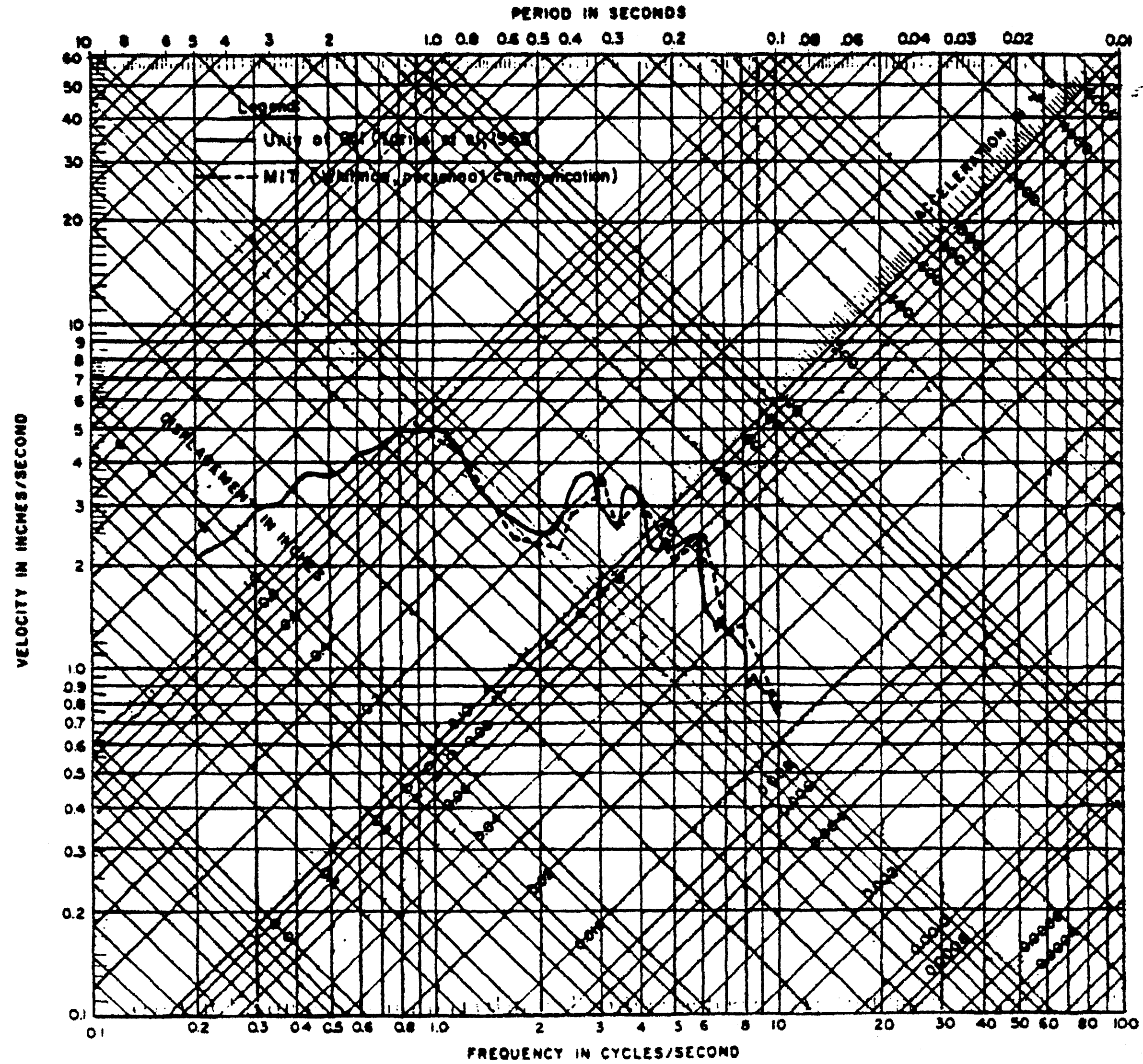
RECOMMENDED RESPONSE SPECTRA FOR HORIZONTAL
VIBRATORY GROUND MOTIONS OF
MAXIMUM PROBABLE EARTHQUAKE (SMALLER EARTHQUAKE)
(0.08 g)
FOR SEVERAL DAMPING RATIOS



COMPARISON OF RESPONSE SPECTRA FOR
ZERO DAMPING

WMAI 68-192

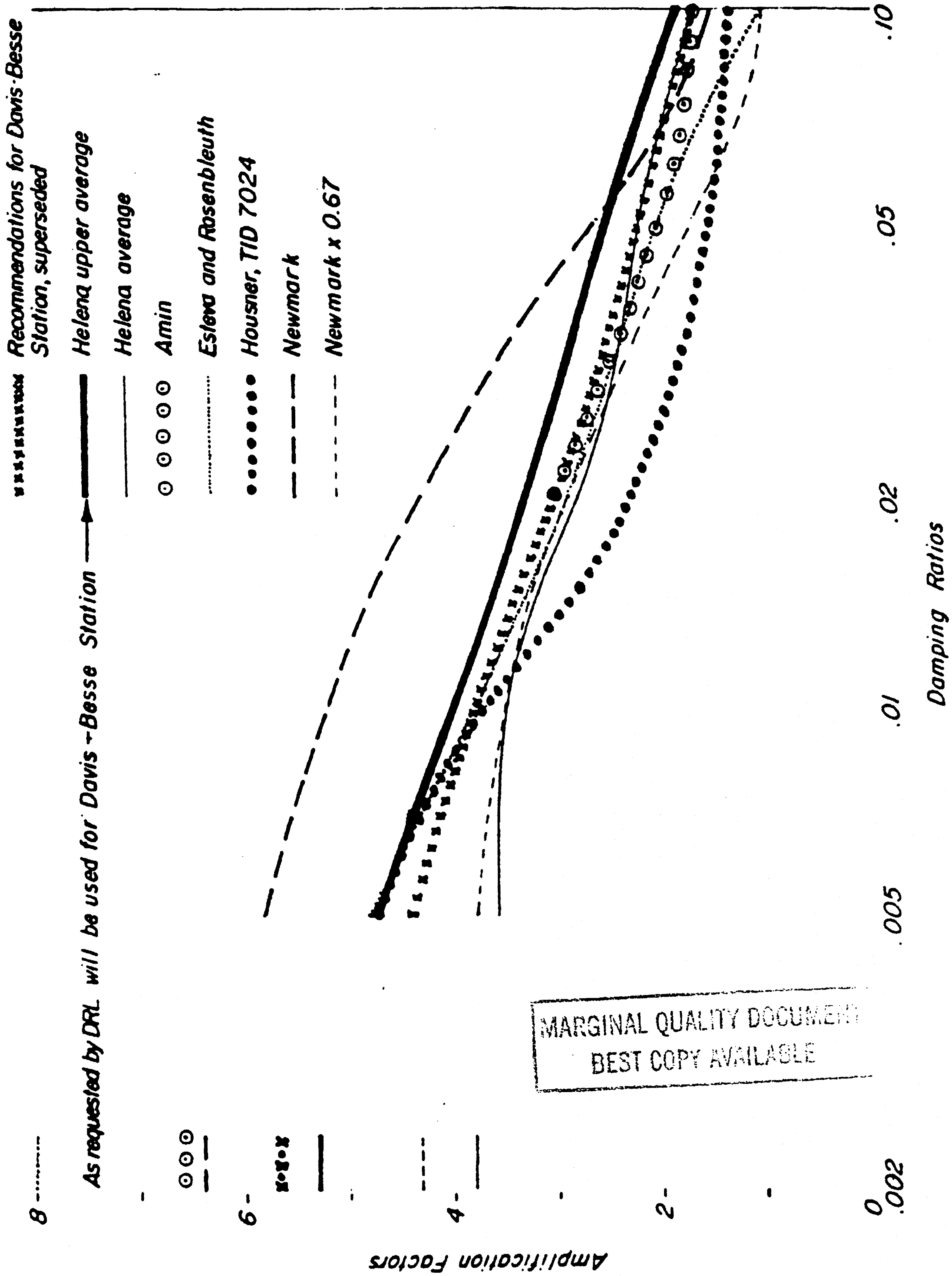
FIGURE 2C.3-7
REVISION 0
JULY 1982



COMPARISON OF TWO INDEPENDENTLY CALCULATED RESPONSE SPECTRA OF THE EAST-WEST COMPONENT OF THE 31 OCTOBER 1935 HELENA EARTHQUAKE

WMAI 68-192

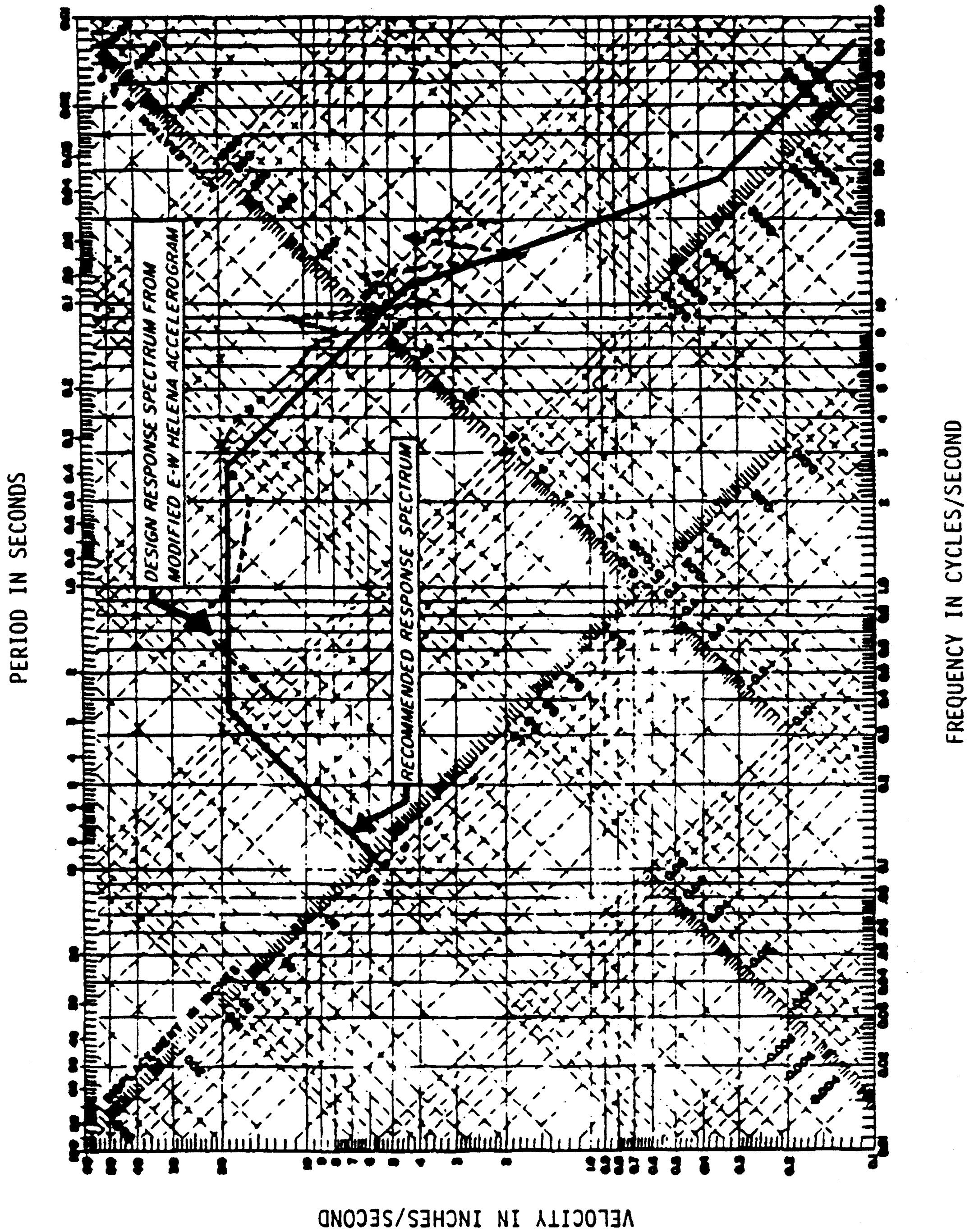
FIGURE 2C.3-8
REVISION 0
JULY 1982



AMPLIFICATION FACTORS FOR SPECTRAL ACCELERATION

WMAI 68-192

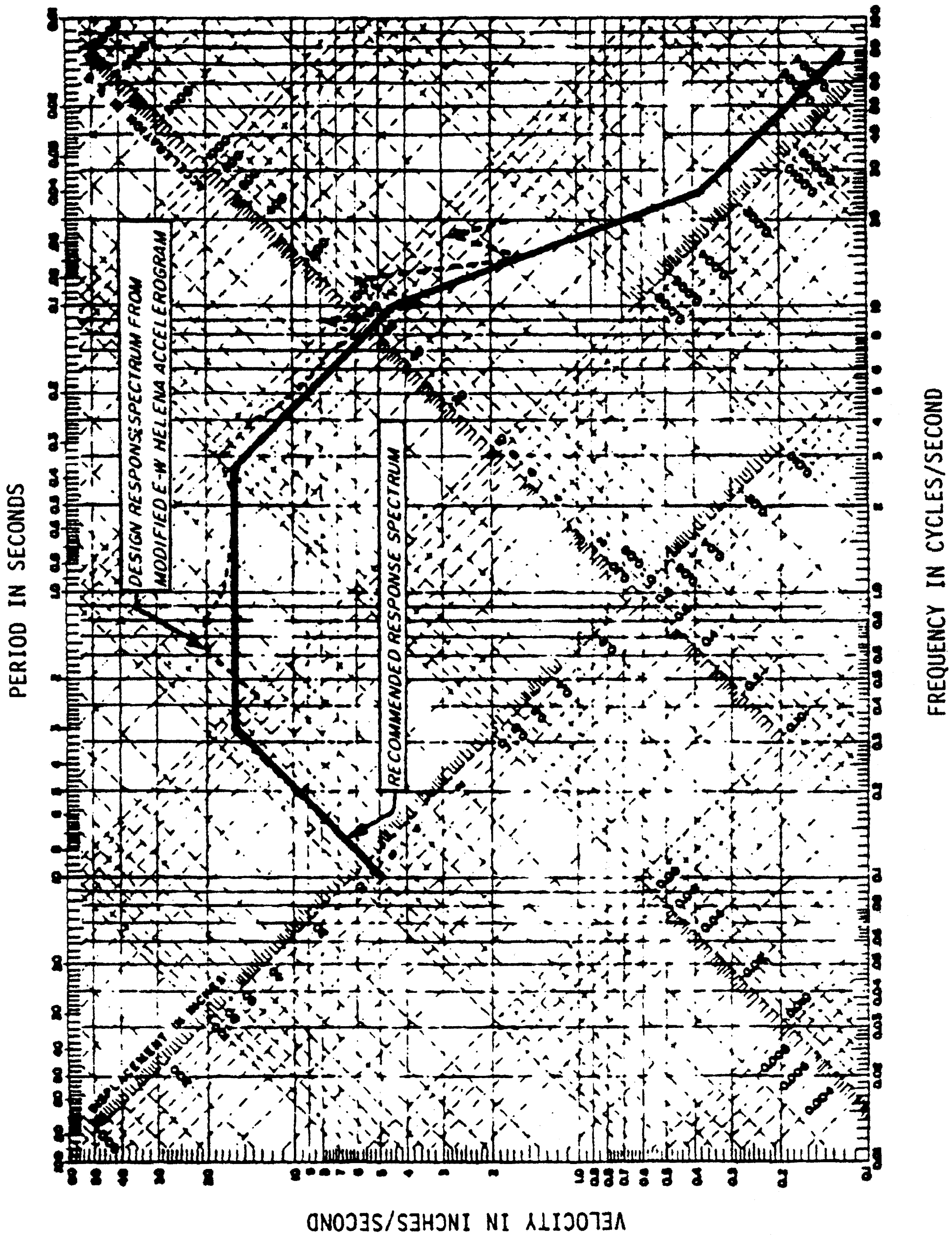
FIGURE 2C.3-9
REVISION 0
JULY 1982



COMPARISON OF RECOMMENDED AND DESIGN RESPONSE SPECTRA-$\lambda = 0.00$

WMAI 68-192

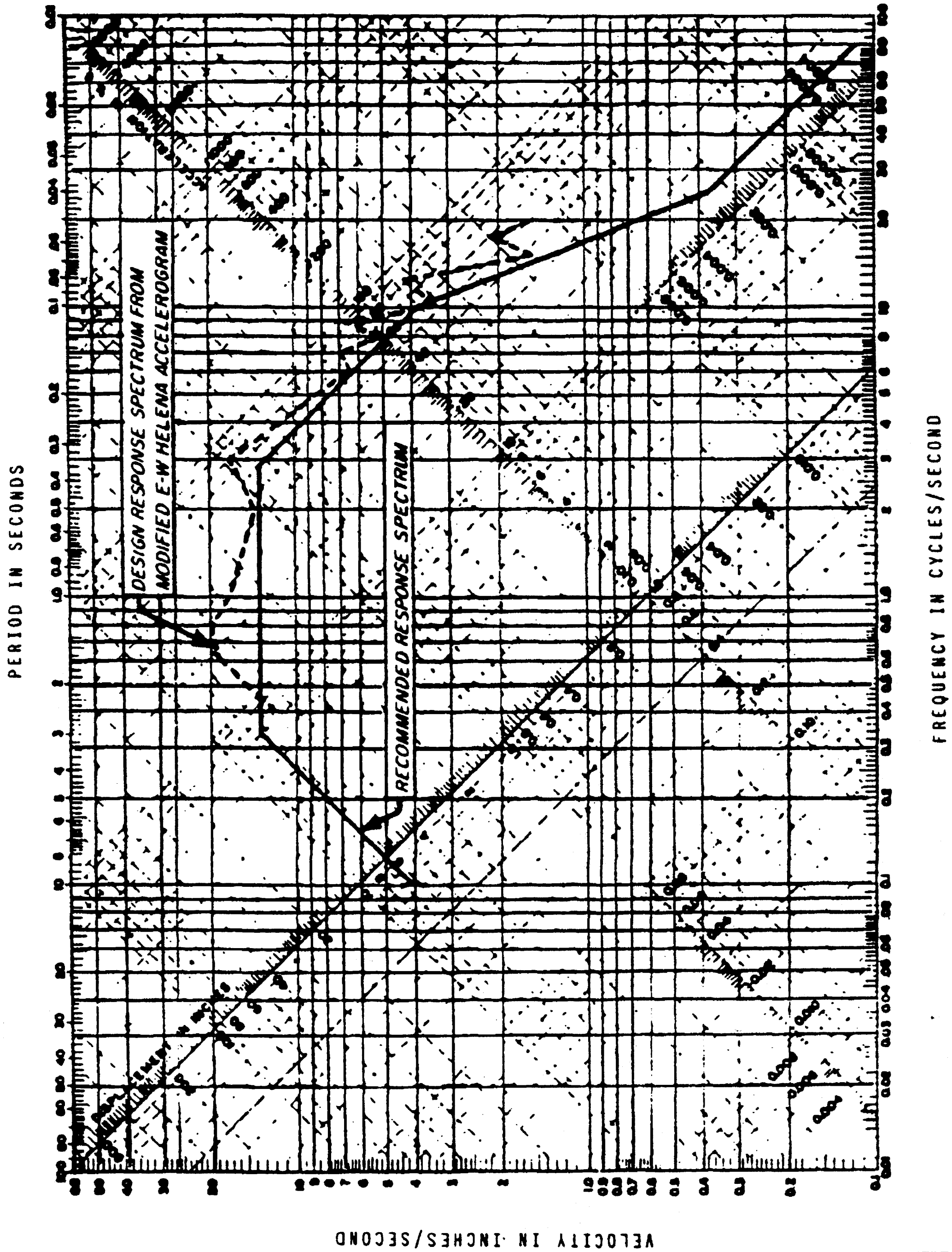
FIGURE 2C.3-10
REVISION 0
JULY 1982



COMPARISON OF RECOMMENDED AND DESIGN RESPONSE SPECTRA-λ=0.005

WMAI 68-192

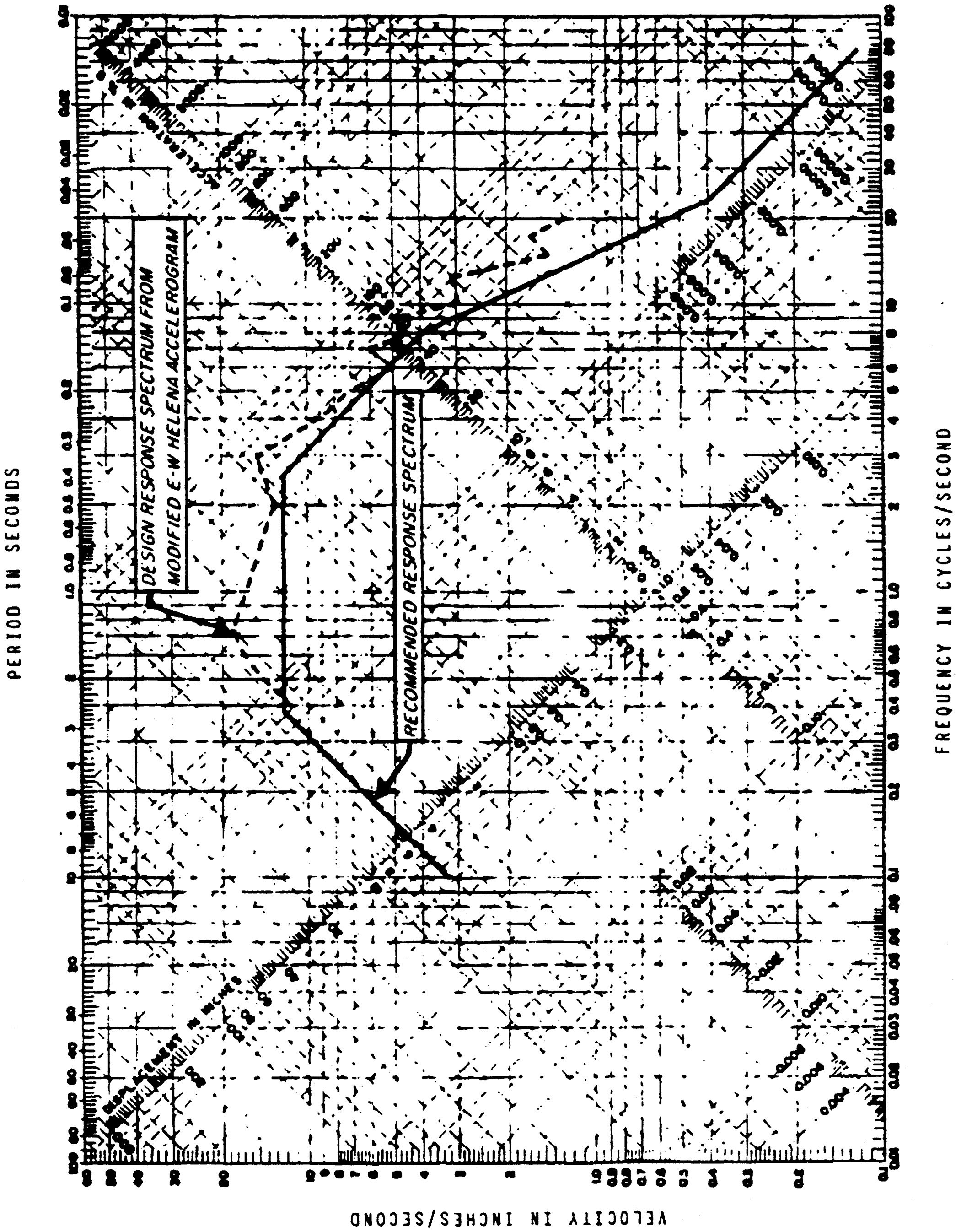
FIGURE 2C.3-11
REVISION 0
JULY 1982



COMPARISON OF RECOMMENDED AND DESIGN RESPONSE SPECTRA-λ=0.01

WMAI 68-192

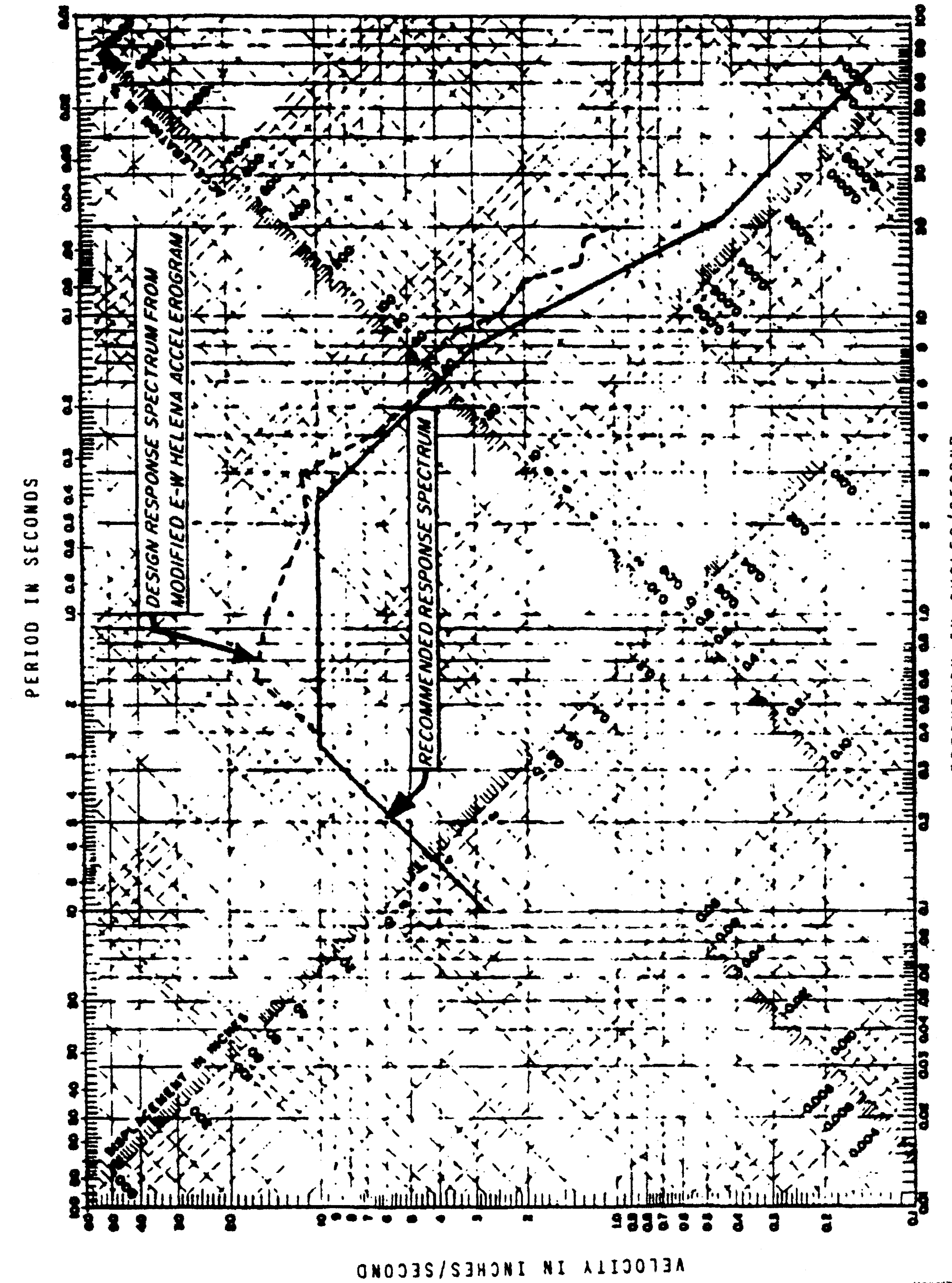
FIGURE 2C.3-12
REVISION 0
JULY 1982



COMPARISON OF RECOMMENDED AND DESIGN RESPONSE SPECTRA-λ=0.02

WMAI 68-192

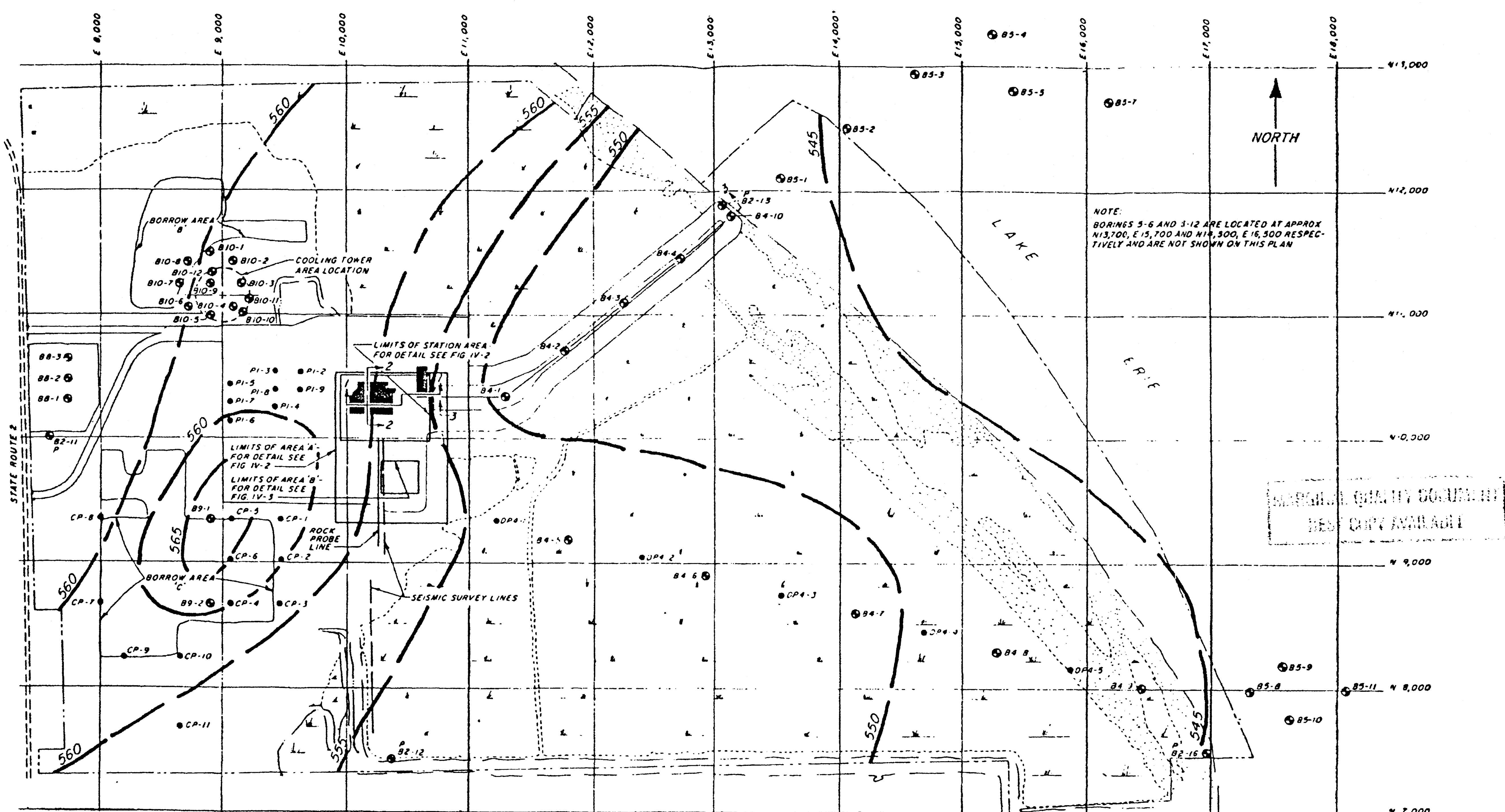
FIGURE 2C.3-13
REVISION 0
JULY 1982



COMPARISON OF RECOMMENDED AND DESIGN RESPONSE SPECTRA-λ=0.05

WMAI 68-192

FIGURE 2C.3-14
REVISION 0
JULY 1982



NOTES

1. GENERAL LAYOUT OBTAINED FROM DWG NO. C-1, REV 7, PREPARED BY BECHTEL COMPANY, GAITHERSBURG, MARYLAND, DATED 12 AUG 70.
2. COORDINATES SHOWN ARE REFERENCED TO PROJECT COORDINATE SYSTEM.
3. BORINGS AND PROBES LOCATED ON THIS FIGURE ARE THOSE WHICH ARE OUTSIDE THE LIMITS OF AREA 'A'.
4. ELEVATIONS ARE REFERENCED TO INTERNATIONAL GREAT LAKES DATUM (IGLD).
5. SEE FIG. IV-8, IV-9 AND IV-10 FOR GENERALIZED GEOLOGIC PROFILES.

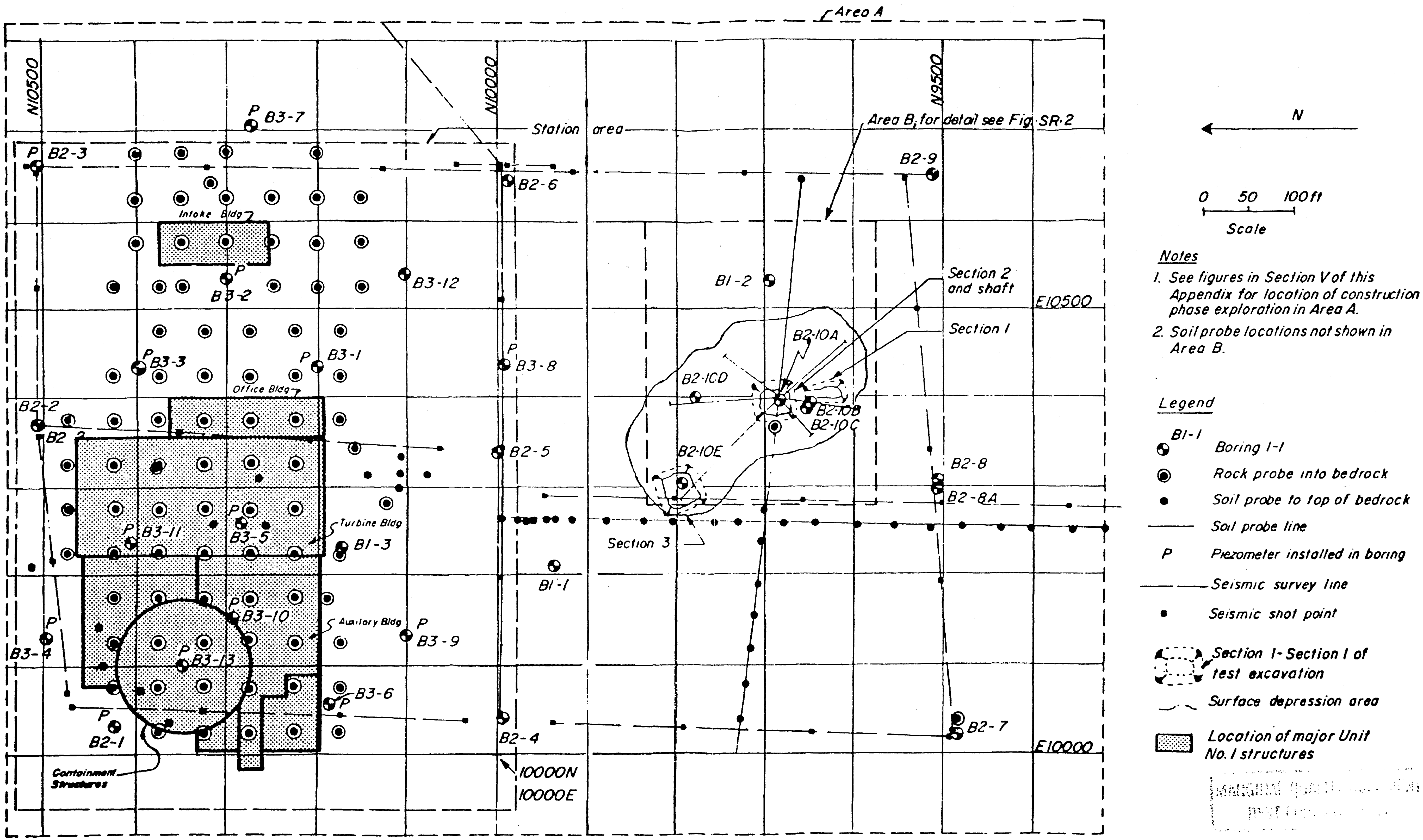
LEGEND

- CP-1 SAMPLED CONSTRUCTION PROBE NO. 1
- PI-5 SAMPLED PROBE 1-5
- DP4-2 UNSAMPLED PROBE 4-2
- B4-1 BORING 4-1
- 550 INFERRED BEDROCK ELEVATION
- P PIEZOMETER LOCATION
- MARSH DEPOSIT
- SAND DEPOSIT
- MAJOR STATION AREA STRUCTURE
- SITE AREA PROPERTY LINE
- LAKE ERIE SHORELINE
- PROFILE 1-1

LOCATIONS OF BORINGS, PROBES, AND SEISMIC SURVEY LINES IN SITE AREA

FIGURE 2C.4-1
REVISION 0
JULY 1982

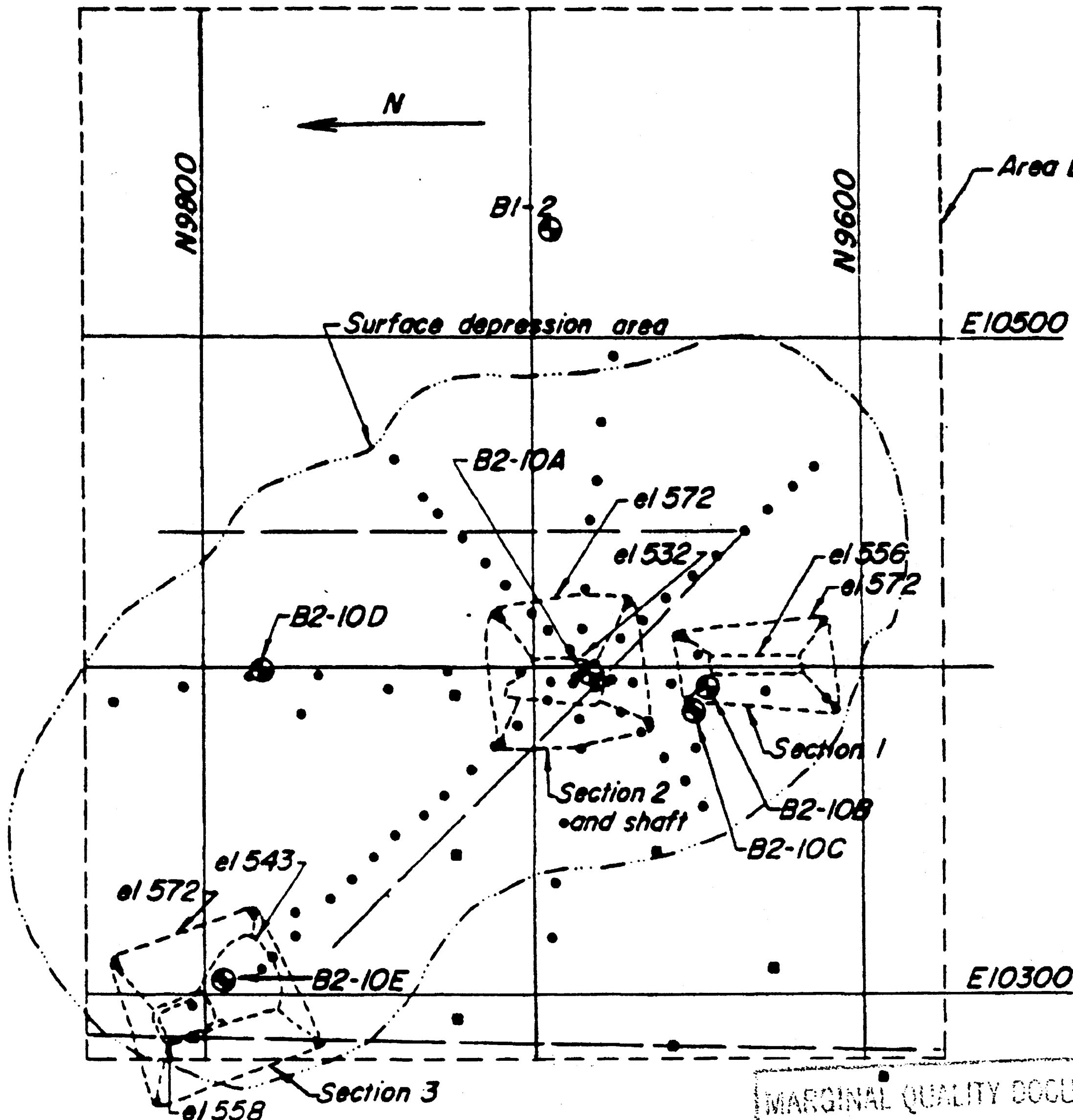
WMAI 68-192



LOCATION OF PRECONSTRUCTION EXPLORATION IN AREA A

WMAI 68-192

FIGURE 2C.4-2
REVISION 0
JULY 1982



NOTES

1. Elevations referenced to International Great Lakes Datum (IGLD)
2. Coordinates shown are referenced to project coordinate system

0 50
Scale, ft

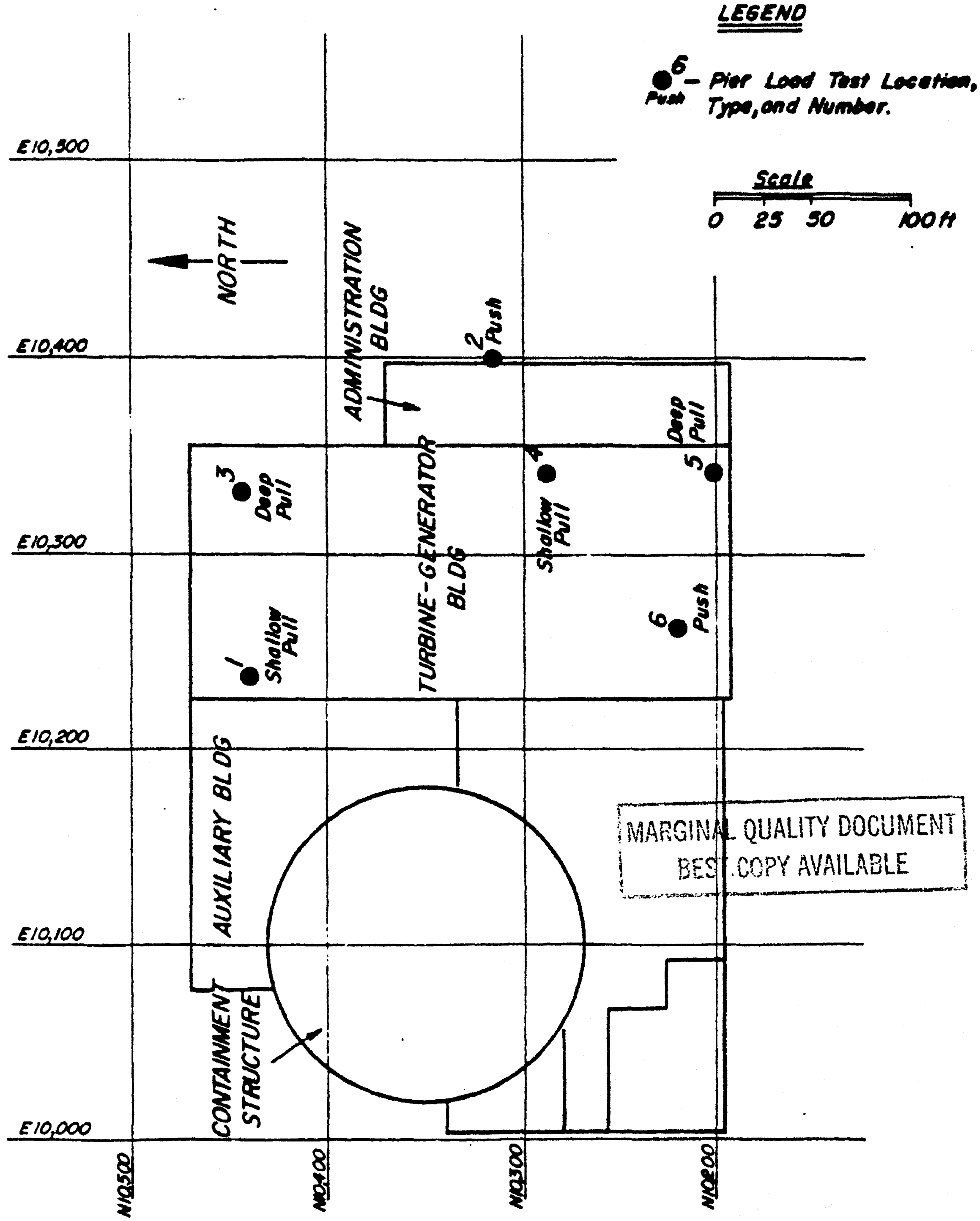
LEGEND

- B2-10A Boring 2-10A
- Soil probe to top of bedrock
- Seismic shot point
- Seismic survey line
- el 558 - el bottom of excavation
- el 574 - el top of excavation
- Section 1 - Section 1 of test excavation

LOCATION OF PRECONSTRUCTION EXPLORATION IN AREA B

WMAI 68-192

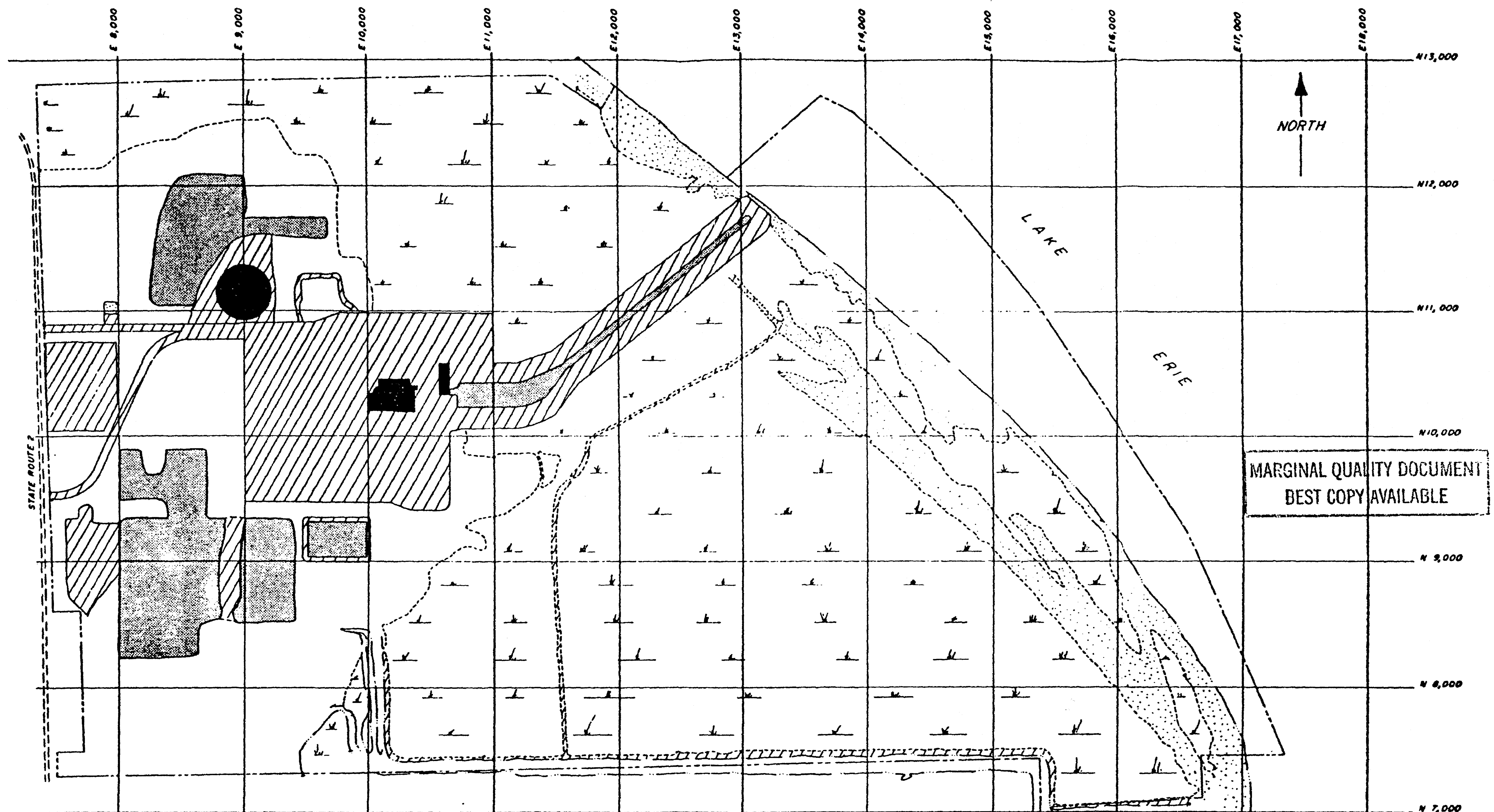
FIGURE 2C.4-3
REVISION 0
JULY 1982



LOCATION OF TEST PIERS
BEDROCK SOCKET - CONCRETE BOND SHEAR TESTS

WMAI 68-192

FIGURE 2C.4-4
REVISION 0
JULY 1982



NOTES:

1. GENERAL LAYOUT OBTAINED FROM DWG NO. C-1, REV 7, PREPARED BY BECHTEL COMPANY, GAITHERSBURG, MARYLAND, DATED 12 AUG 70.
2. COORDINATES SHOWN ARE REFERENCED TO PROJECT COORDINATE SYSTEM.

LEGEND

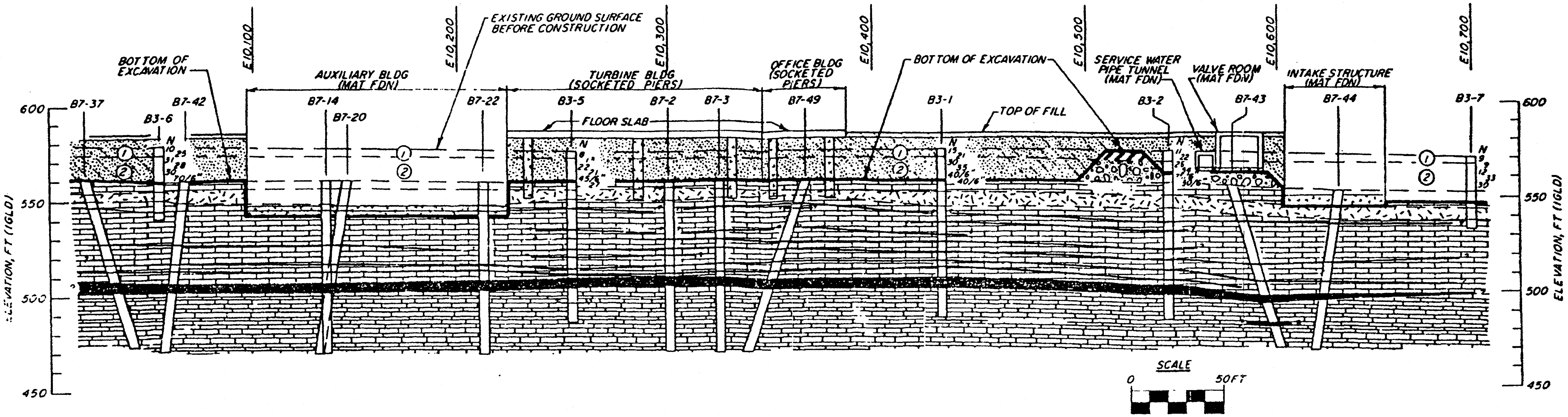
- GLACIOLACUSTRINE DEPOSIT
- MARSH DEPOSIT
- SAND DEPOSIT
- FILL
- EXCAVATION AREA
- MAJOR STATION AREA STRUCTURE
- SITE AREA PROPERTY LINE
- LAKE ERIE SHORELINE

SCALE: 1 IN.= 800FT (APPROX)

MAP OF SITE AREA SURFICIAL SOILS

WMAI 68-192

FIGURE 2C.4-5
REVISION 0
JULY 1982



GENERALIZED DESCRIPTION OF GEOLOGIC MATERIALS

- GLACIOLACUSTRINE DEPOSIT – STIFF BROWN AND GRAY VARVED SILTY CLAY
- TILL DEPOSIT – VERY STIFF GRAVELLY, SANDY AND SILTY CLAY
- DOLOMITE – MEDIUM HARD TO HARD, GRAY TO BROWN, LAMINATED TO THIN BEDDED, ANHYDRITE LAMINAE OCCUR OCCASIONALLY BELOW EL 530
- DOLOMITE – HARD, LIGHT GRAY TO GRAY, MEDIUM TO MASSIVE BEDDED, UPPER 2 TO 4 FT CONTAINS 0.001 TO 0.003 FT DIA ANHYDRITE INCLUSIONS
- SHALE – MEDIUM HARD, DARK GREEN TO BLACK, THIN TO MEDIUM BEDDED, DISCONTINUOUS ABOVE EL 510
- DOLOMITE AND ANHYDRITE – INTERBEDDED, MEDIUM HARD TO HARD, BLUE GRAY TO BROWN, THIN BEDDED
- COMPACTED FILL – GLACIOLACUSTRINE AND TILL DEPOSITS
- COMPACTED FILL – GRANULAR MATERIAL

BUILDING SYMBOLS

- MAT FOOTING FOUNDATION
- PIER FOOTING FOUNDATION
- FLOOR SLAB AND ROOF

NOTES

1. ELEVATIONS SHOWN ARE REFERENCED TO INTERNATIONAL GREAT LAKES DATUM (IGLD).
2. FOR LOCATION OF PROFILE; SEE FIG. IV-1
3. APPROX GROUNDWATER LEVEL BEFORE CONSTRUCTION EL 571.
4. EXTENT OF GLACIOLACUSTRINE AND TILL DEPOSIT EXCAVATED IN THE STATION AREA AND REPLACED WITH COMPACTED GRANULAR BACKFILL IS SHOWN AS 1 AND 2 RESPECTIVELY.

SUMMARY OF ENGINEERING PROPERTIES OF BEDROCK

BORING NO.	NO OF DETERMINATIONS	ROCK CORE RECOVERY		RQD[1]	
		RANGE	AVE	RANGE	AVE
B3-1	17	62-100	91	0-55	16
B3-2	10	50-106	94	0-78	31
B3-5	10	100	100	12-76	43
B3-6	4	89-100	97	8-50	29
B3-7	4	98-103	100	54-92	71
B7-2	11	96-100	99	17-100	64
B7-3	11	97-100	99	13-99	65
B7-14	10	94-100	99	20-92	54
B7-20	14	75-100	95	0-87	60
B7-22	10	92-100	99	22-95	54
B7-37	18	65-100	97	0-100	47
B7-42	15	50-100	92	0-100	50
B7-43	12	86-100	99	0-90	39
B7-44	13	100	100	11-100	48
B7-49	13	93-100	99	18-100	60

(1) RQD - ROCK QUALITY DESIGNATION

LEGEND

B3-6 ← BORING NUMBER

N 10 25 31 28 – STANDARD PENETRATION RESISTANCE

GENERALIZED WEST-EAST GEOLOGIC PROFILE I-I

WMAI 68-192

FIGURE 2C.4-6
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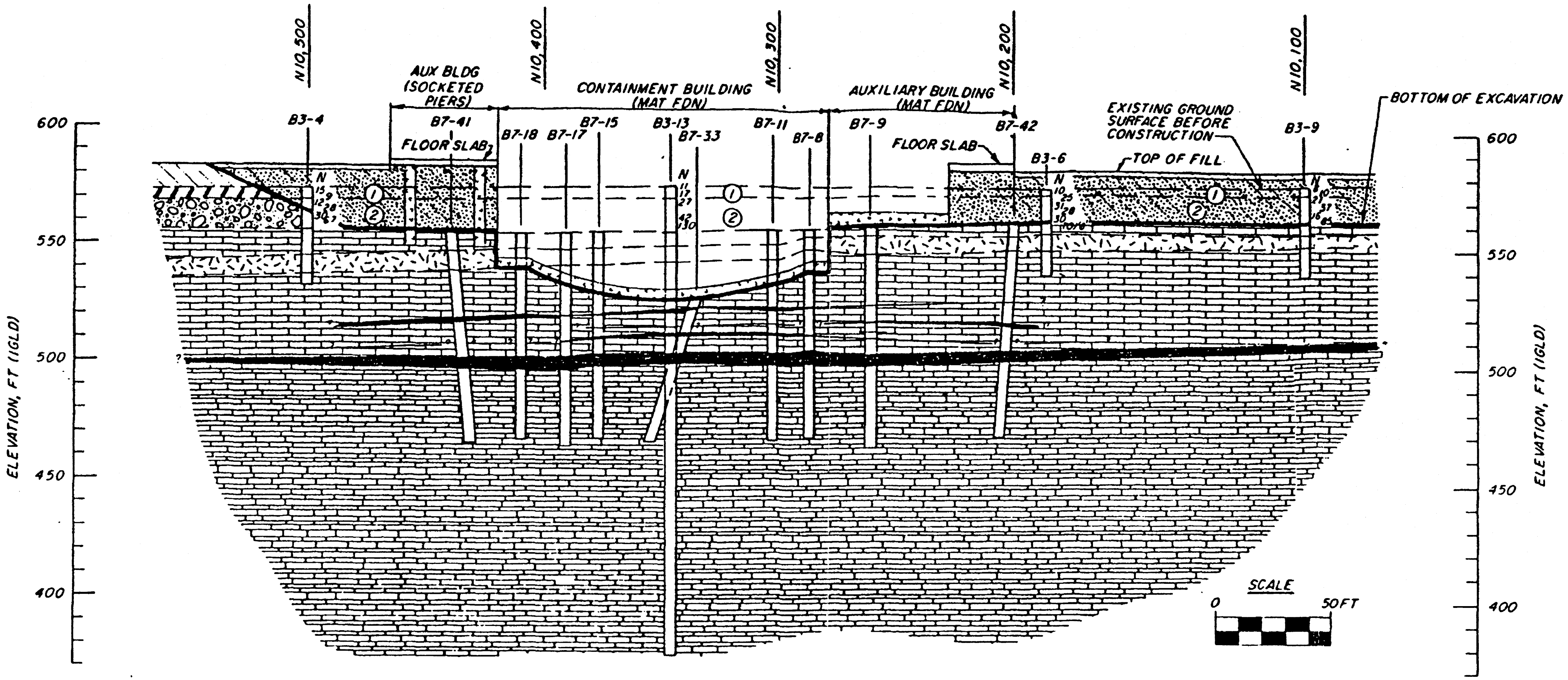

GENERALIZED DESCRIPTION OF GEOLOGIC MATERIALS

- GLACIOLACUSTRINE DEPOSIT - STIFF BROWN AND GRAY VARVED SILTY CLAY
- TILL DEPOSIT - VERY STIFF GRAVELLY, SANDY AND SILTY CLAY
- DOLOMITE - MEDIUM HARD TO HARD, GRAY TO BROWN, LAMINATED TO THIN BEDDED, ANHYDRITE LAMINAE OCCUR OCCASIONALLY BELOW EL 530
- DOLOMITE - HARD, LIGHT GRAY TO GRAY, MEDIUM TO MASSIVE BEDDED, UPPER 2 TO 4 FT CONTAINS 0.001 TO 0.003 FT DIA ANHYDRITE INCLUSIONS
- SHALE - MEDIUM HARD, DARK GREEN TO BLACK, THIN TO MEDIUM BEDDED, DISCONTINUOUS ABOVE EL 510
- DOLOMITE AND ANHYDRITE - INTERBEDDED, MEDIUM HARD TO HARD, BLUE GRAY TO BROWN, THIN BEDDED
- COMPACTED FILL - GLACIOLACUSTRINE AND TILL DEPOSITS
- COMPACTED FILL - GRANULAR MATERIAL

BUILDING SYMBOLS

- MAT FOOTING FOUNDATION
- PIER FOOTING FOUNDATION
- FLOOR SLAB

NOTES

1. ELEVATIONS SHOWN ARE REFERENCED TO INTERNATIONAL GREAT LAKES DATUM (IGLD).
2. FOR LOCATION OF PROFILE, SEE FIG. IV-1.
3. APPROX. GROUNDWATER LEVEL BEFORE CONSTRUCTION EL 571.
4. EXTENT OF GLACIOLACUSTRINE AND TILL DEPOSIT EXCAVATED IN THE STATION AREA AND REPLACED WITH COMPACTED GRANULAR BACKFILL IS SHOWN AS (1) AND (2) RESPECTIVELY.

SUMMARY OF ENGINEERING PROPERTIES OF BEDROCK

BORING NO	NO OF DETER-MINATIONS	ROCK CORE RECOVERY		RQD(1)	
		RANGE	AVE	RANGE	AVE
B3-4	4	86-100	96	0-33	23
B3-6	4	89-100	97	8-50	29
B3-9	4	92-100	97	24-60	42
B3-13	20	95-103	100	44-94	68
B7-8	11	80-100	97	21-60	41
B7-9	12	92-100	99	8-85	45
B7-11	10	92-100	99	22-69	47
B7-15	11	98-100	100	0-90	57
B7-17	10	80-100	97	9-74	43
B7-18	10	97-100	100	15-82	37
B7-33	10	93-100	99	12-96	67
B7-41	12	95-100	99	12-92	66
B7-42	15	50-100	92	0-100	50

(1) RQD - ROCK QUALITY DESIGNATION

LEGEND

B3-4 ← BORING NUMBER

N 15, 9, 12, 28 } - STANDARD PENETRATION RESISTANCE

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GENERALIZED NORTH-SOUTH GEOLOGIC PROFILE 2-2

FIGURE 2C.4-7
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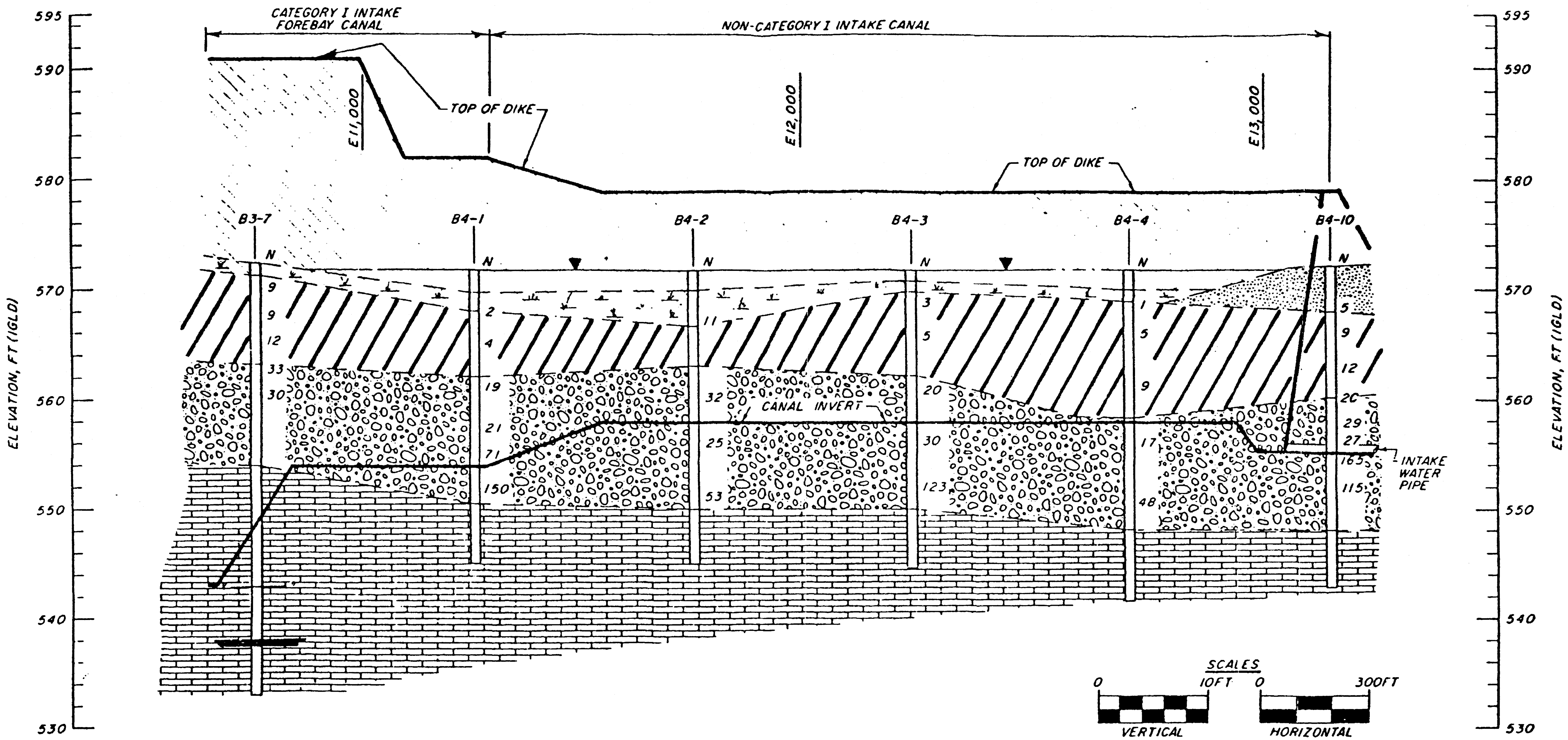

GENERALIZED DESCRIPTION OF GEOLOGIC MATERIALS

COMPACTED FILL - GLACIOLACUSTRINE AND TILL DEPOSITS

ORGANIC DEPOSIT - DECAYED ORGANIC MATTER, PEAT AND GRAY TO BLACK CLAY, TRACE SAND

GLACIOLACUSTRINE DEPOSIT - STIFF BROWN AND GRAY VARVED SILTY CLAY

TILL DEPOSIT - VERY STIFF TO HARD GRAVELLY, SANDY AND SILTY CLAY

DOLOMITE - MEDIUM HARD TO HARD, GRAY TO BROWN LAMINATED TO THIN BEDDED

SAND DEPOSIT - FINE SILTY SAND, TRACE COARSE SAND AND FINE GRAVEL

SHALE - MEDIUM HARD, DARK GREEN TO BLACK, THIN TO MEDIUM BEDDED, DISCONTINUOUS

NOTES

1. ELEVATIONS REFERENCED TO INTERNATIONAL GREAT DATUM (IGLD).
2. FOR LOCATION OF PROFILE, SEE FIG. IV-1
3. ORGANIC DEPOSIT EXCAVATED BENEATH DIKE BEFORE PLACEMENT AND COMPACTION OF DIKE FILL.

SUMMARY OF ENGINEERING PROPERTIES OF BEDROCK

BORING NO.	NO OF DETER-MINATIONS	ROCK CORE RECOVERY		RQD(1)	
		RANGE	AVE	RANGE	AVE
B3-7	4	98-103	100	54-92	71
B4-1	1	58	58	0	0
B4-2	1	64	64	10	10
B4-3	1	54	54	6	6
B4-4	2	57-100	78.5	0-20	10
B4-10	1	74	74	0	0

(1) RQD - ROCK QUALITY DESIGNATION

LEGEND

B4-1 ← BORING NUMBER

N

2
4
19
} - STANDARD PENETRATION RESISTANCE

WMAI 68-192

GENERALIZED GEOLOGIC PROFILE 3-3

FIGURE 2C.4-8
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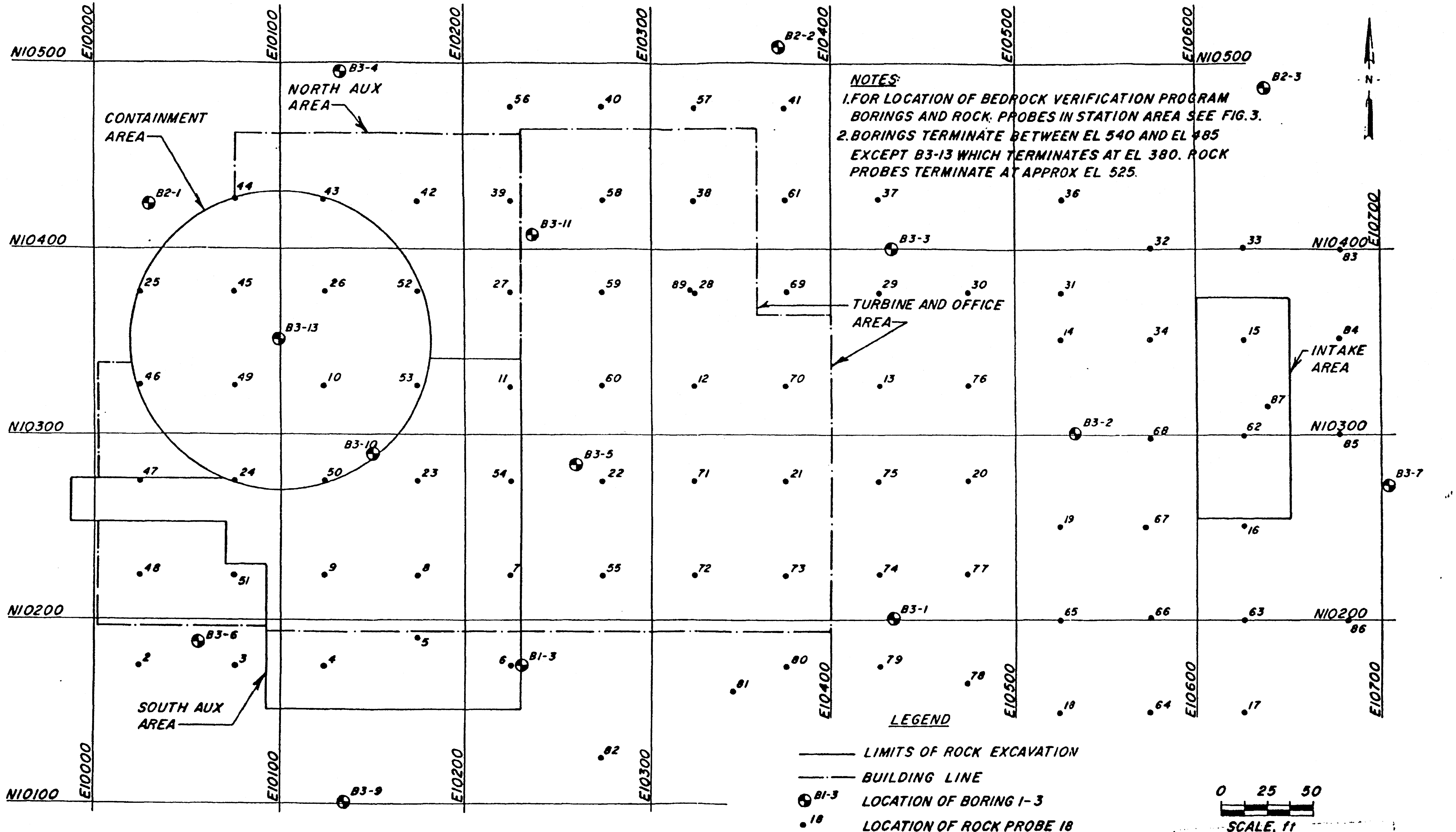

PRE-CONSTRUCTION NX-BORINGS AND ROCK PROBES
IN THE STATION AREA

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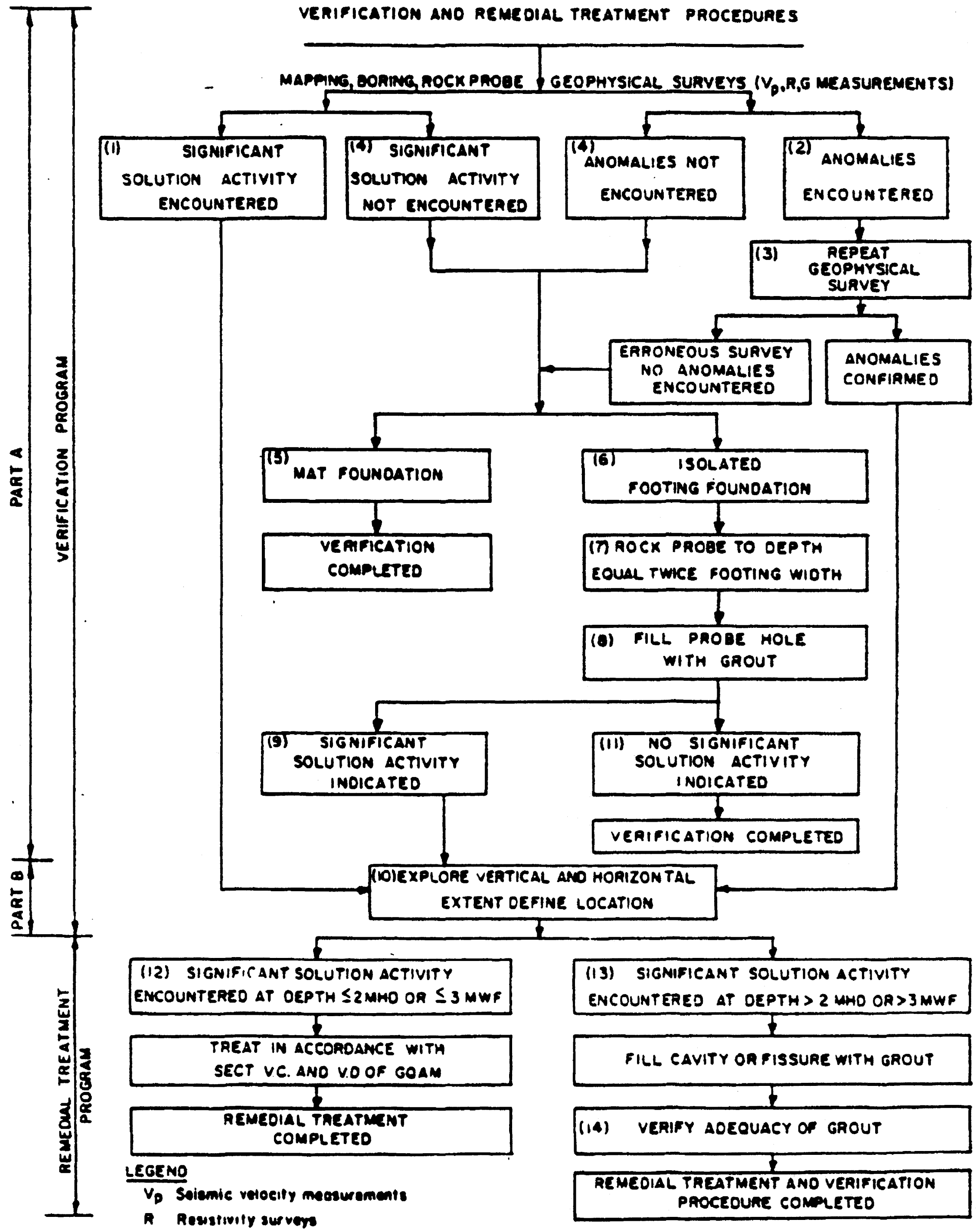
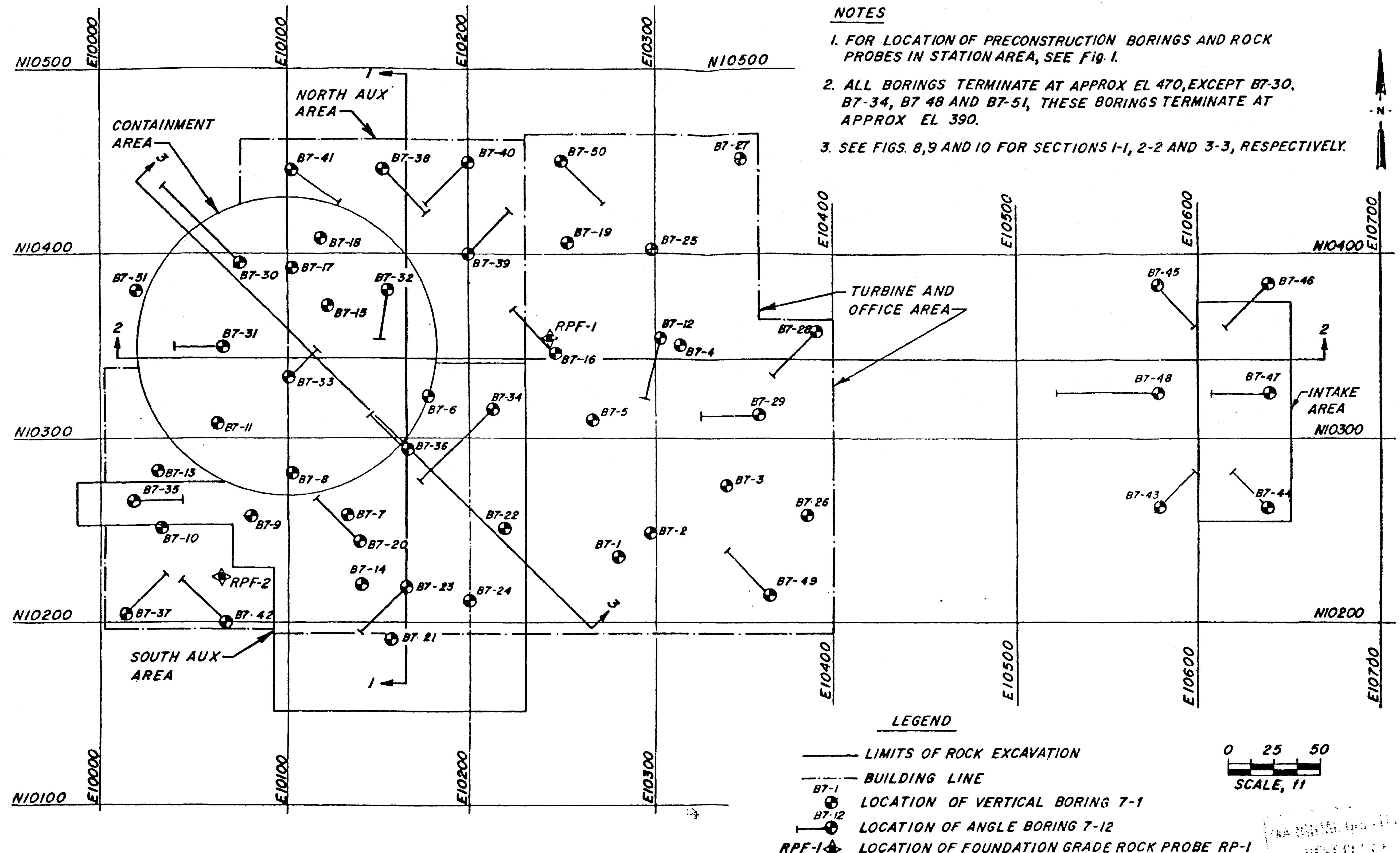
FIGURE 2C.5-1



LEGEND

- V_p Seismic velocity measurements
- R Resistivity surveys
- G Gravity surveys
- MHD Maximum horizontal dimension of significant cavity
- MWF Maximum width of significant fissure
- (2) Refers to number in parenthesis in section 3.4 in text of GOAM

FLOW CHART FOR VERIFICATION AND REMEDIAL TREATMENT PROGRAMS



BEDROCK VERIFICATION NX-BORINGS AND ROCK PROBES IN THE STATION AREA

FIGURE 2C.5-3

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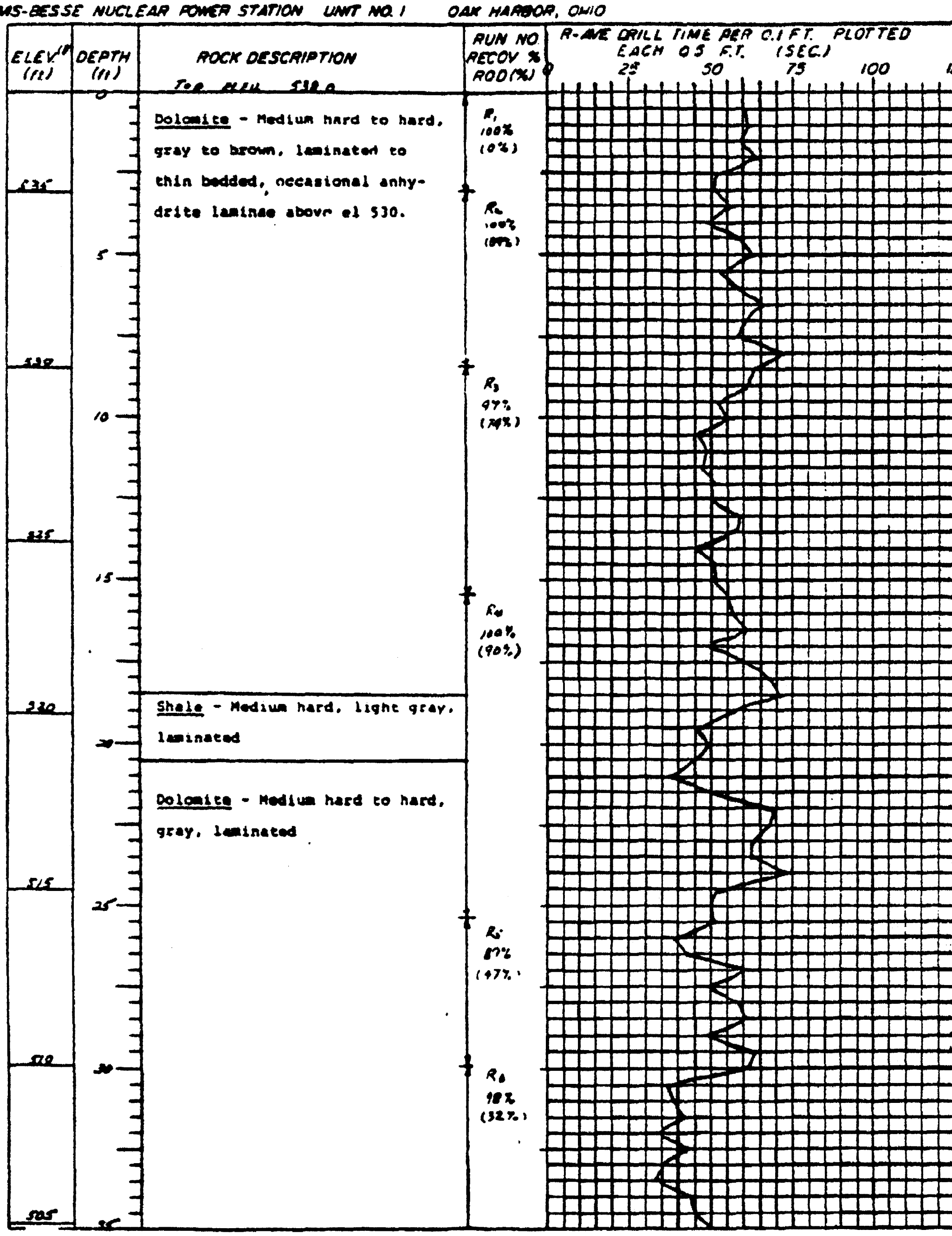
DAVIS-BESSE NUCLEAR POWER STATION UNIT NO. 1 OAK HARBOR, OHIO



NOTES:
(1) ELEVATIONS REFERENCED TO INTERNATIONAL GREAT LAKES DATUM (IGLD)

DESIGN BORING LOG - BORING NO. B7-30
BEARING N45W ORIENTATION 20°

FIGURE 2C.5-4

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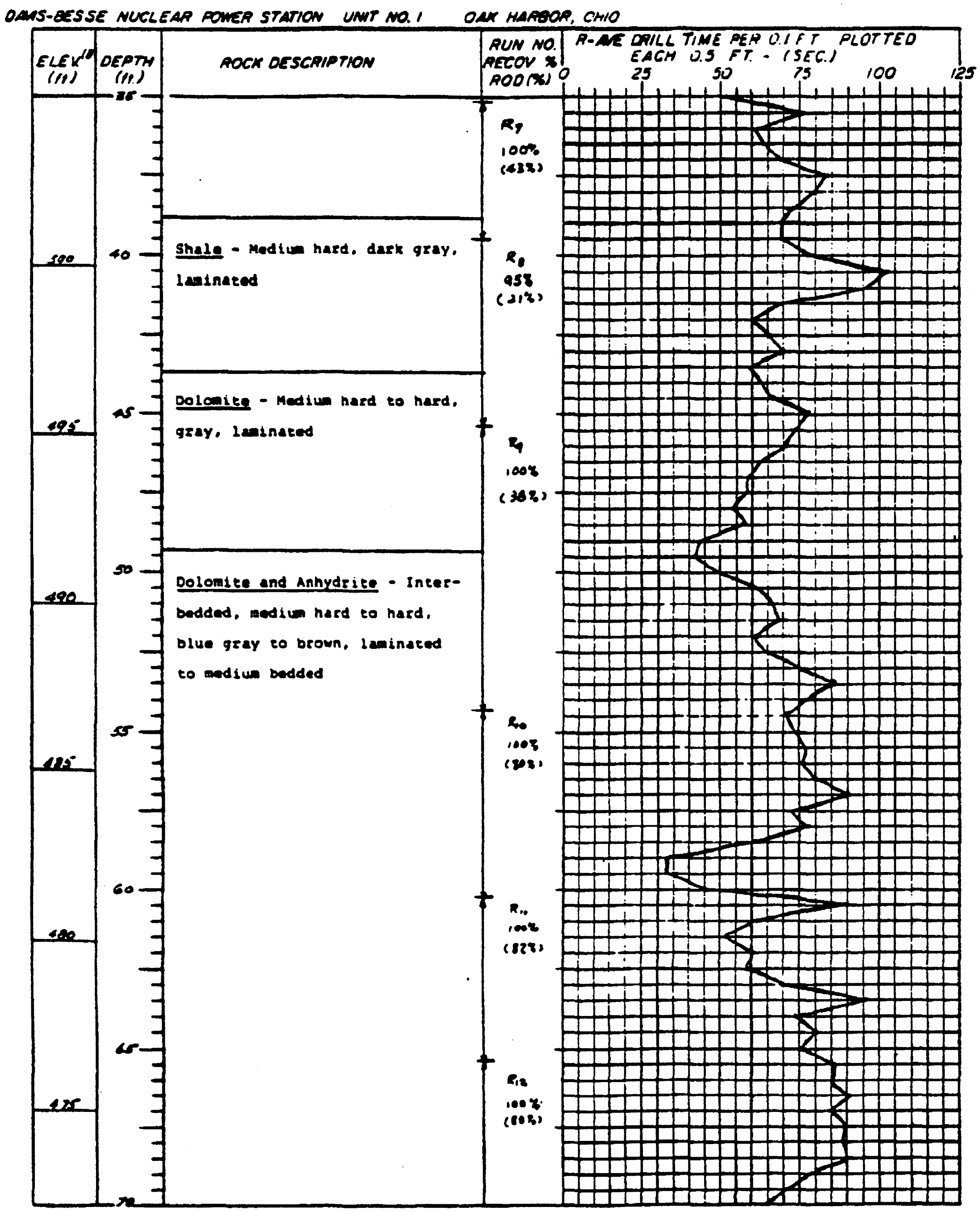

NOTES:
(1) ELEVATIONS REFERENCED TO INTERNATIONAL GREAT LAKES DATUM (IGLD)

DESIGN BORING LOG - BORING NO. B7-30 (CON'T)
BEARING N45W ORIENTATION 20°

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FIGURE 2C.5-5

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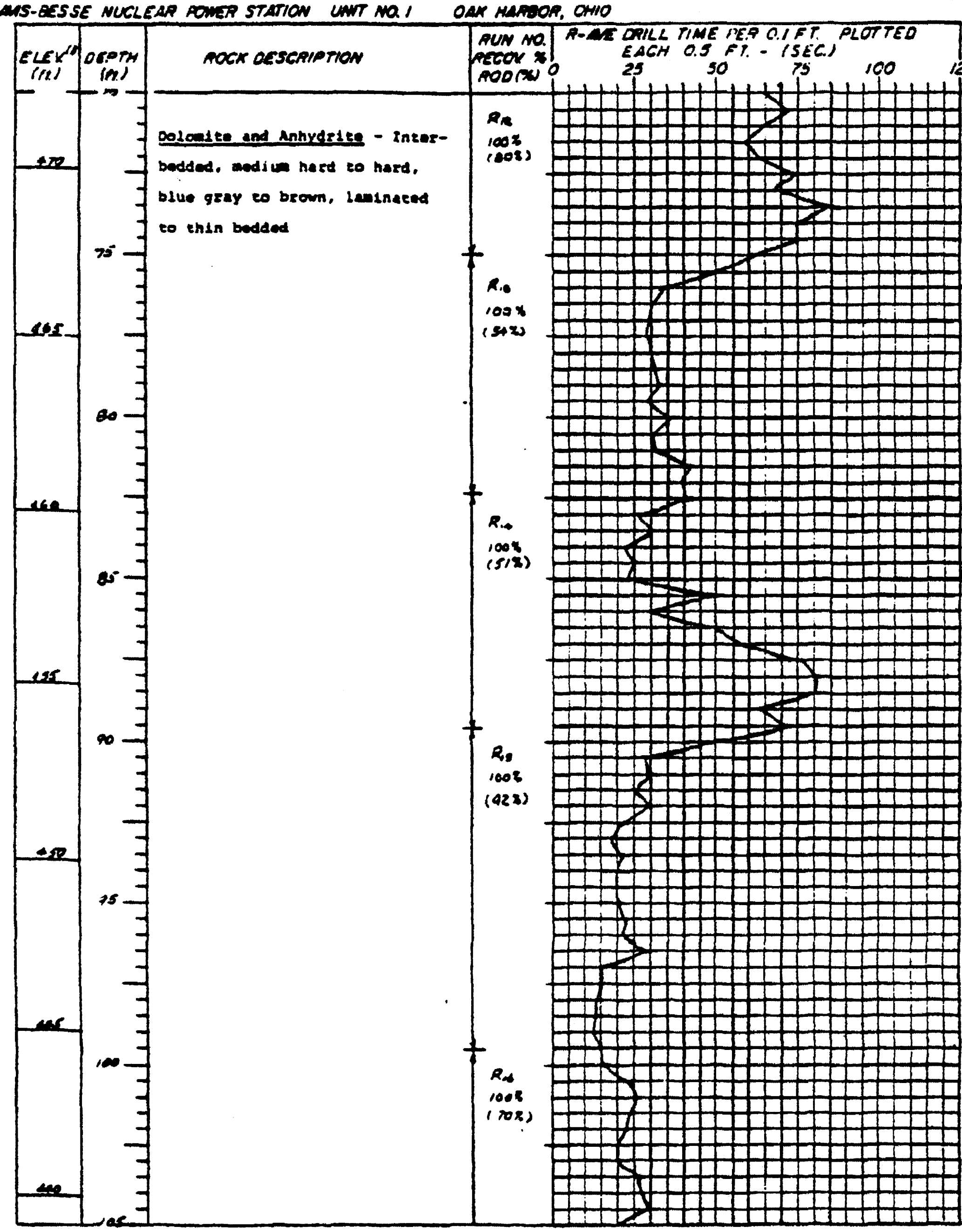
DAVIS-BESSE NUCLEAR POWER STATION UNIT NO. 1 OAK HARBOR, OHIO



NOTES.
(1) ELEVATIONS REFERENCED TO INTERNATIONAL GREAT LAKES DATUM (IGLD)

DESIGN BORING LOG - BORING NO. B2-30 (CON'T)
BEARING N45W ORIENTATION 30°

FIGURE 2C.5-6

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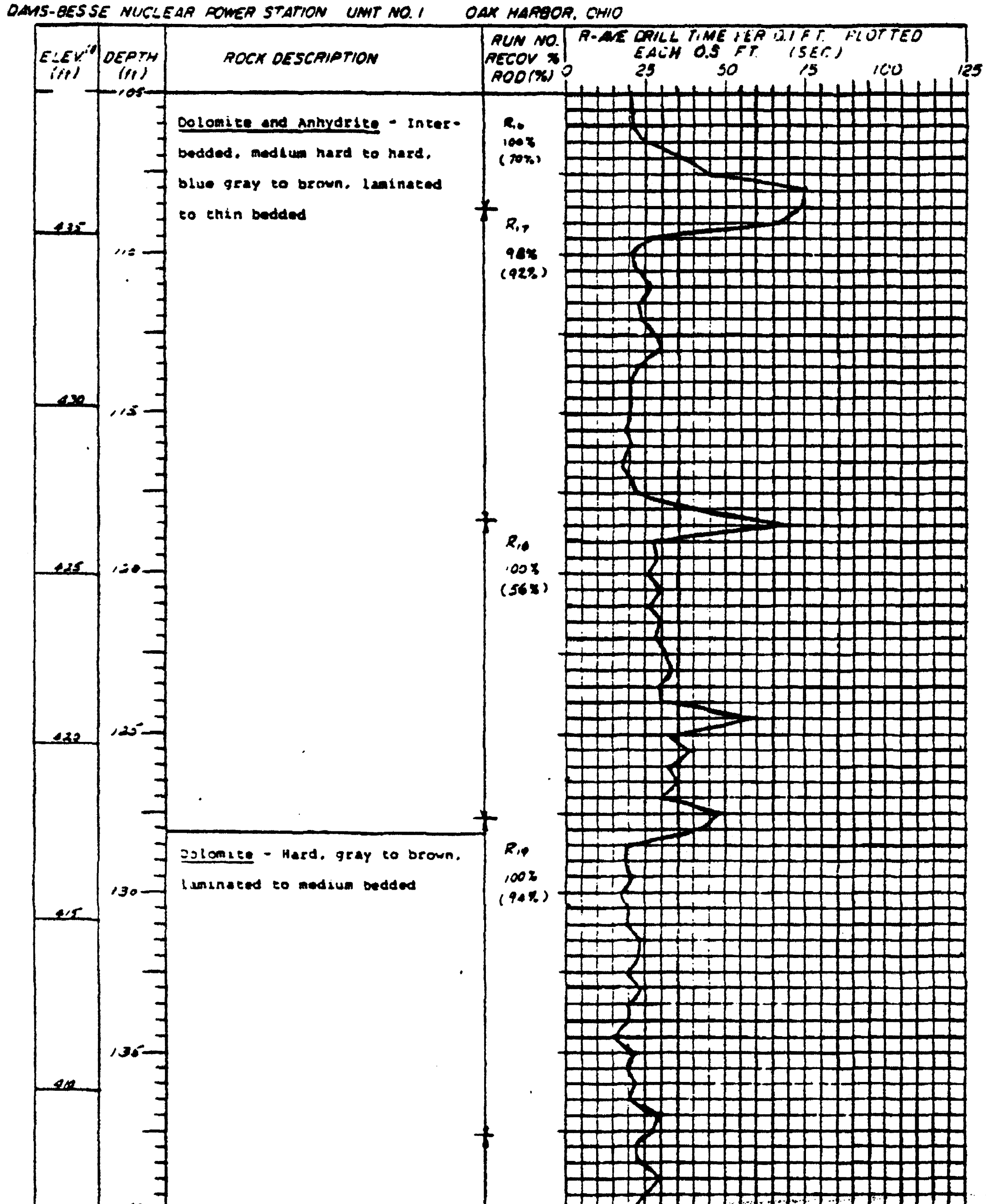
DAVIS-BESSE NUCLEAR POWER STATION UNIT NO. 1 OAK HARBOR, OHIO



NOTES.
(1) ELEVATIONS REFERENCED TO INTERNATIONAL GREAT LAKES DATUM (IGLD)

DESIGN BORING LOG - BORING NO. B7-30 (CON'T)
BEARING N45W ORIENTATION 20°

FIGURE 2C.5-7

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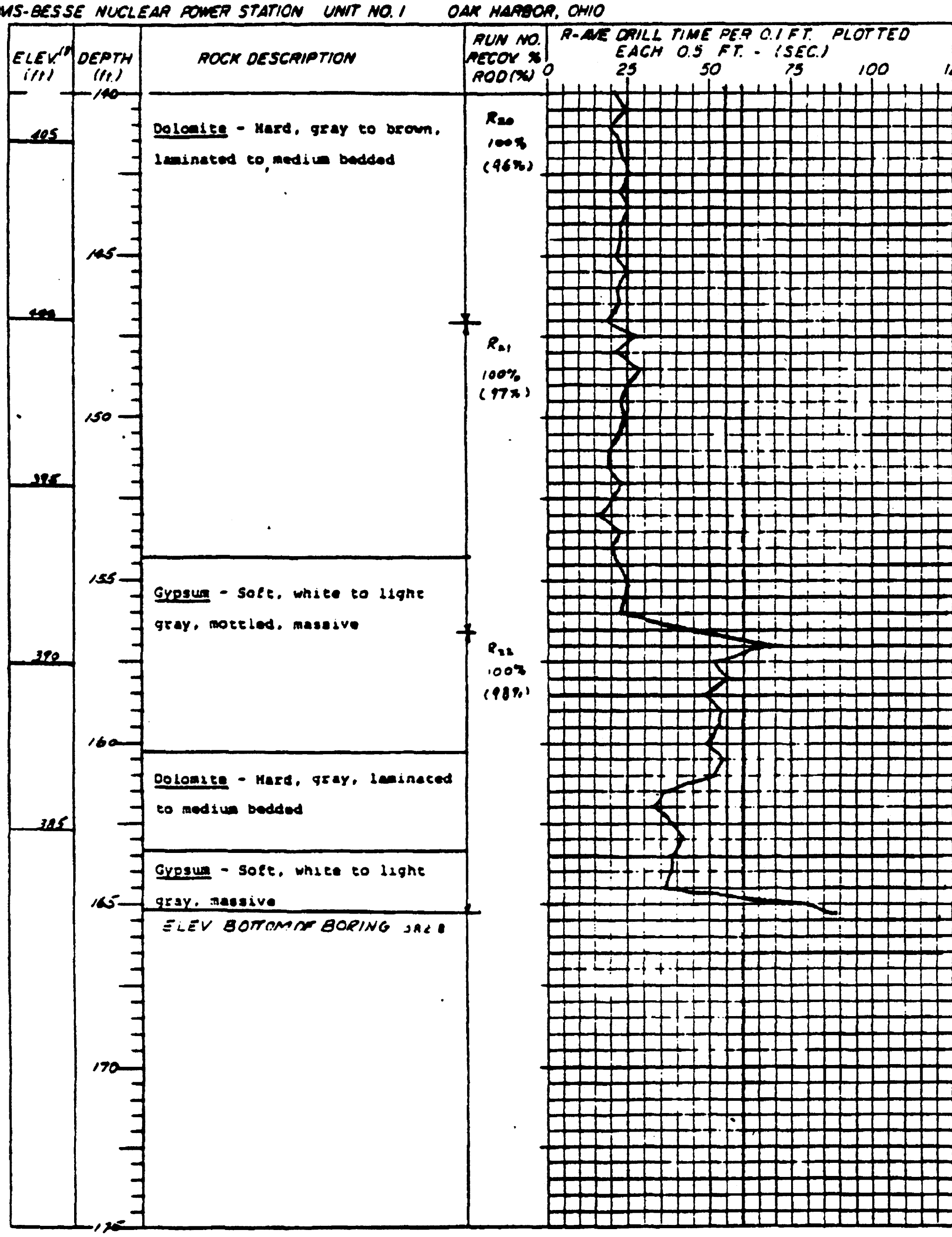
DAVIS-BESSE NUCLEAR POWER STATION UNIT NO. 1 OAK HARBOR, OHIO



NOTES
(1) ELEVATIONS REFERENCED TO INTERNATIONAL GREAT LAKES DATUM (IGLD)

DESIGN BORING LOG - BORING NO. B7-32(CON'T)
BEARING N45W ORIENTATION 20°

FIGURE 2C.5-8

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Ground Surface El. 538.0

Project: Davis-Besse Nuclear Power Plant Boring B7-10 Water Table El. ______

Core No.	Recovery %	RQD %	Rock type	Top el	Thickness ft	Classification and Description
R1	100	0	Dol	538	0.1	Medium hard, dark grey to buff DOLOMITE BRECCIA, with dolomite clusts in calcareous matrix, 1% vugs, 1/32" - 1/16"
			Anh/Dol		2.9	Medium hard to hard, light grey to white, irregularly laminated to mottled ANHYDRITE AND DOLOMITE, with a few 1/32" laminae of white gypsum, 1% partial solution fissures 1/16" - 1/4"
R2	100	89	Anh/Dol	535.2	5.4	Same as Above
R3	97	74	Anh/Dol	530.1	6.8	Same as Above
R4	100	90	Dol	523.5	2.9	Medium hard, dark grey to light grey, irregularly laminated to thinly bedded DOLOMITE with one stringer of gypsum cutting bedding perpendicularly 4" x 1/4", 1% partial solution fissures 1/32" - 1/16"
	.		Sh/Dol		2.0	Medium hard, light grey to greenish grey, irregularly laminated to mottled DOLOMITE SHALE, with one 1/4" laminae of satin spar gypsum
			Dol		3.1	Hard to medium hard, light grey to dark grey, irregularly laminated DOLOMITE, with 1% gypsum 1/32" and 1% partial solution fissures 1/32" - 1/8"
			Dol		2.0	Medium hard to hard, light grey, medium bedded to irregularly laminated DOLOMITE
R5	89	47	Dol	514.1	4.0	Hard to medium hard, light grey, medium bedded to irregularly laminated argillaceous DOLOMITE, with 1% gypsum 1/32"
R6	98	32	Dol	509.9	2.8	Same as Above
					2.4	Hard to medium hard, dark grey to light grey, irregularly laminated to thin bedded DOLOMITE, with 1% gypsum 1/16" - 1/4", and <1% brown anhydrite inclusions, 1/16" - 1/2"
R7	100	42	Dol	504.9	0.7	Same as Above
			Dol		2.6	Medium hard, light brownish grey to white, irregularly laminated to thin bedded, DOLOMITE, with 1% gypsum, 1/2" to 1/4" and 1% partial solution fissures
			Dol/Sh		1.0	Medium hard, light grey, irregularly laminated to mottled SHALY DOLOMITE, with 1% gypsum 1/32" - 1/16"
R8	95	21	Dol/Sh	500.9	3.2	Same as Above
			Dol		2.3	Hard to medium hard, light grey, irregularly laminated DOLOMITE, with 1% gypsum 1/32"
R9	100	38	Dol	495.4	2.6	Same as Above
			Dol		1.2	Hard, light brownish grey, medium bedded argillaceous DOLOMITE
			Anh		5.2	Hard, light bluish grey, medium bedded ANHYDRITE, <1% partial solution fissures 1/32" - 1/16"
R10	100	80	Anh	486.9	3.2	Same as Above
			Ls		2.4	Medium hard, buff, irregularly laminated LIMESTONE, with 5% dolomite inclusions 1/32" - 1/16"

ROCK CLASSIFICATION DATA SHEET

FIGURE 2C.5-9

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Ground Surface El. 538.0

Project: Davis-Besse Nuclear Power Plant   Boring B7-30   Water Table El. ______

Core No.	Recovery %	RQD %	Rock type	Top el	Thickness ft	Classification and Description
			Anh		0.4	Hard, light bluish-grey, medium bedded ANHYDRITE, with 1% anhydrite inclusions 1/8" - 1/4"
R11	100	82	Anh	481.1	5.1	Same as Above
R12	100	80	Anh	476.5	1.1	Same as Above
			Dol/Anh		2.9	Hard to medium hard, light grey to dark blue, medium bedded to irregularly laminated, DOLOMITE and ANHYDRITE
			Dol		2.8	Medium hard to hard, light brownish grey to light bluish grey, irregularly laminated to medium bedded DOLOMITE with 1% gypsum 1/16" - 1/2"
			Anh		2.8	Hard to medium hard, light bluish grey to buff, irregularly laminated to mottled, DOLOMITE, with 1% brown anhydrite 1/16" - 1/8"
R13	100	54	Anh	467.5	1.5	Same as Above
			Dol		3.2	Medium hard, light brownish grey, irregularly laminated DOLOMITE, with 5% gypsum 1/8" - 1/4", 1% vugs 1/32" - 1/64"
			Anh		2.7	Medium hard, light grey, irregularly laminated DOLOMITE, with 1% brown anhydrite 1/32" and 1% vugs 1/32" - 1/64"
R14	100	51	Anh	460.5	0.7	Same as Above
			Dol		2.7	Hard, light brownish grey, irregularly laminated argillaceous DOLOMITE
			Anh/Dol		3.7	Medium hard to hard, light to dark grey, irregularly laminated to mottled ANHYDRITE and DOLOMITE with 1% gypsum 1/16" - 1/2"
			Dol/Anh		0.1	Hard to medium hard, light grey, irregularly laminated DOLOMITE and ANHYDRITE with 1% gypsum 1/32" - 1/8"
R15	100	92	Dol/Anh	453.7	8.5	Same as Above
			Dol		1.5	Hard, light grey, mottled, argillaceous DOLOMITE, with 1% vugs, 1/32" - 1/64"
R16	100	70	Dol	444.3	2.9	Same as Above
			Dol		1.6	Hard, light to dark grey, irregularly laminated argillaceous DOLOMITE
			Anh		4.6	Hard, light grey, irregularly laminated to medium bedded ANHYDRITE, with 1% vugs, 1/16" - 1/64"
R17	98	92	Anh	435.7	2.5	Same as Above
			Dol		6.8	Hard, buff, thick bedded, DOLOMITE, with 1% dark grey shale 1/2" - 1"
R18	100	56	Dol	426.8	9.5	Hard, light grey to buff, medium bedded to thinly laminated DOLOMITE with 1% shale 1/2" - 1/8"
R19	100	94	Dol	417.9	0.1	Same as Above
			Dol		5.6	Hard, light grey, medium bedded to irregularly laminated Stylolitic DOLOMITE, with 1% shale 1/64" - 1/16", 1% anhydrite inclusion 1/32" - 1/64" and 1% vugs 1/64" - 1/32"

ROCK CLASSIFICATION DATA SHEET

FIGURE 2C.5-10

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Ground Surface El. 578.0

Project: Davis-Besse Nuclear Power Plant   Boring B7-10   Water Table El

Core No.	Recovery %	RQD %	Rock type	Top el	Thickness ft	Classification and Description
			Dol		4.3	Hard, light grey to buff, thinly laminated to mottled
						DOLOMITE, with 1% vugs 1/64" - 1/16", one gypsum 1/2"
R20	100	96	Dol'	408.5	2.4	Same as Above
			Dol		2.2	Hard, light grey to buff, irregularly laminated to
						thinly bedded DOLOMITE, with 1% shale 1/32" - 1/16"
R21	100	97	Dol	399.5	2.7	Same as Above
			Dol		2.3	Hard, dark to light grey, irregularly laminated
						DOLOMITE with 1% gypsum, 1/16" - 1/4" and 1% vugs
						1/64" - 1/32"
			Dol		1.9	Hard, light grey, irregularly laminated DOLOMITE,
						with 1% white gypsum inclusions, 1/16" - 1/8"
			Gyp		2.4	Low hardness, white to light grey, mottled to
						irregularly laminated GYPSUM with 5% dolomite in-
						clusions 1/4" - 1"
R22	100	98	Gyp	390.8	1.6	Same as Above
			Dol		1.5	Hard, light grey, medium bedded to irregularly
						laminated DOLOMITE with 1% gypsum inclusions 1/8"-1/4"
			Gyp		1.4	Low hardness, light blue to white, medium bedded
						GYPSUM
				382.8		BOTTOM ELEVATION

ROCK CLASSIFICATION DATA SHEET

FIGURE 2C.5-11

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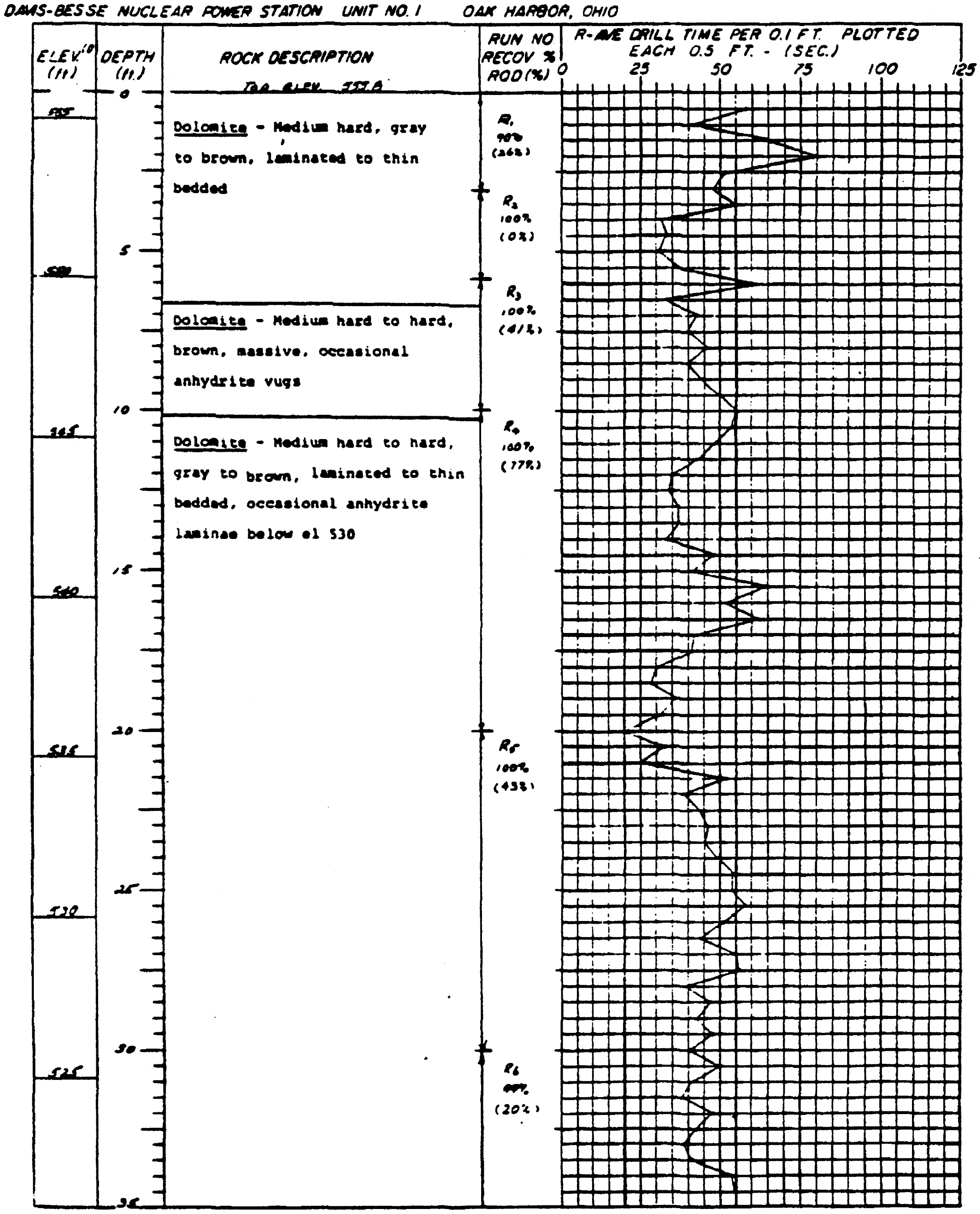
DAVIS-BESSE NUCLEAR POWER STATION UNIT NO. 1 OAK HARBOR, OHIO



NOTES:
(1) ELEVATIONS REFERENCED TO INTERNATIONAL GREAT LAKES DATUM (IGLD)

DESIGN BORING LOG - BORING NO. B 7-51

BEARING — ORIENTATION VERTICAL

FIGURE 2C.5-12

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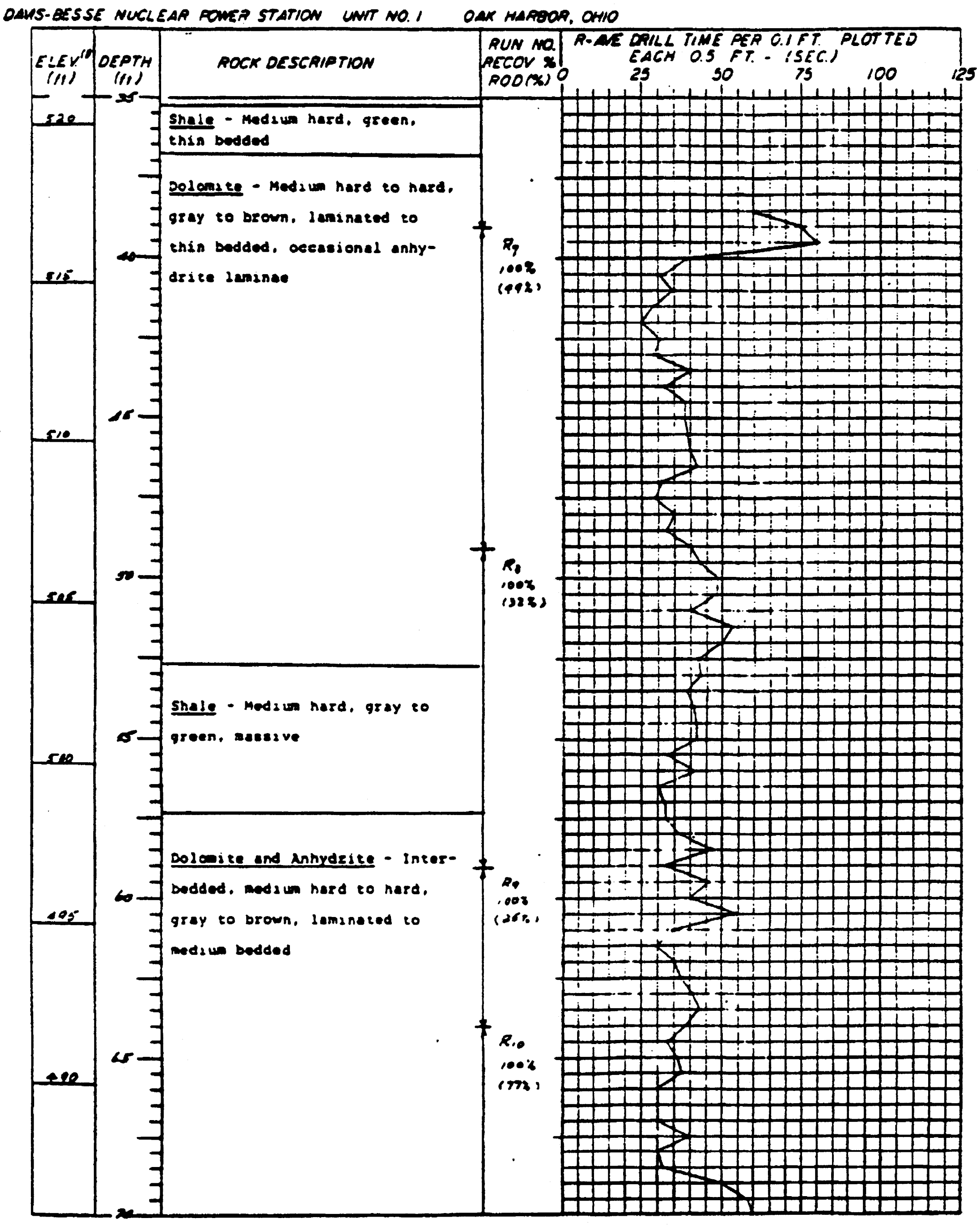

NOTES:
(1) ELEVATIONS REFERENCED TO INTERNATIONAL GREAT LAKES DATUM (IGLD)

DESIGN BORING LOG - BORING NO. B7-51(CON'T)
BEARING — ORIENTATION VERTICAL

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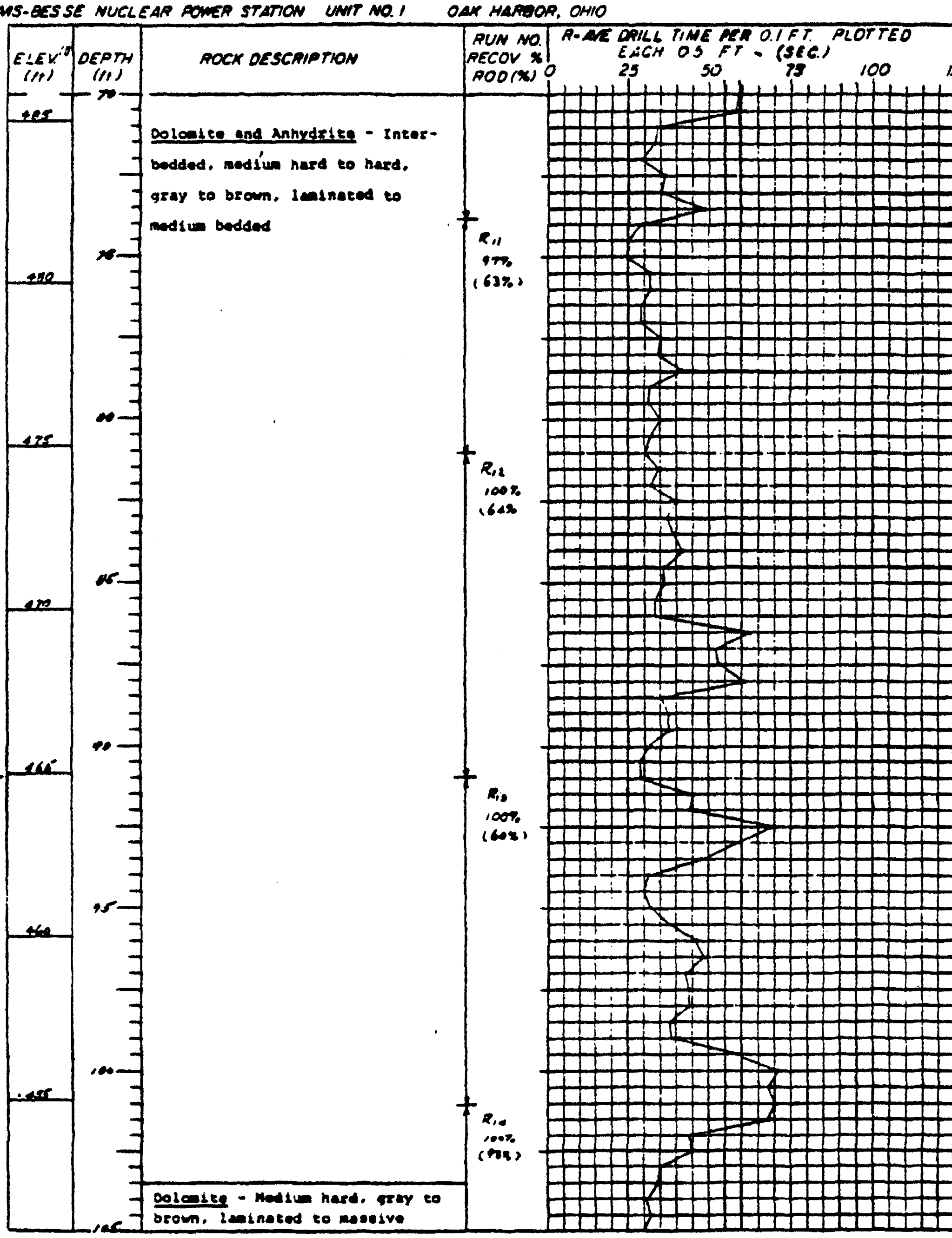
FIGURE 2C.5-13

DAVIS-BESSE NUCLEAR POWER STATION UNIT NO. 1 OAK HARBOR, OHIO



NOTES:
(1) ELEVATIONS REFERENCED TO INTERNATIONAL GREAT LAKES DATUM (IGLD)

DESIGN BORING LOG - BORING NO. B7-S1 (CONT)
BEARING — ORIENTATION VERTICAL

FIGURE 2C.5-14

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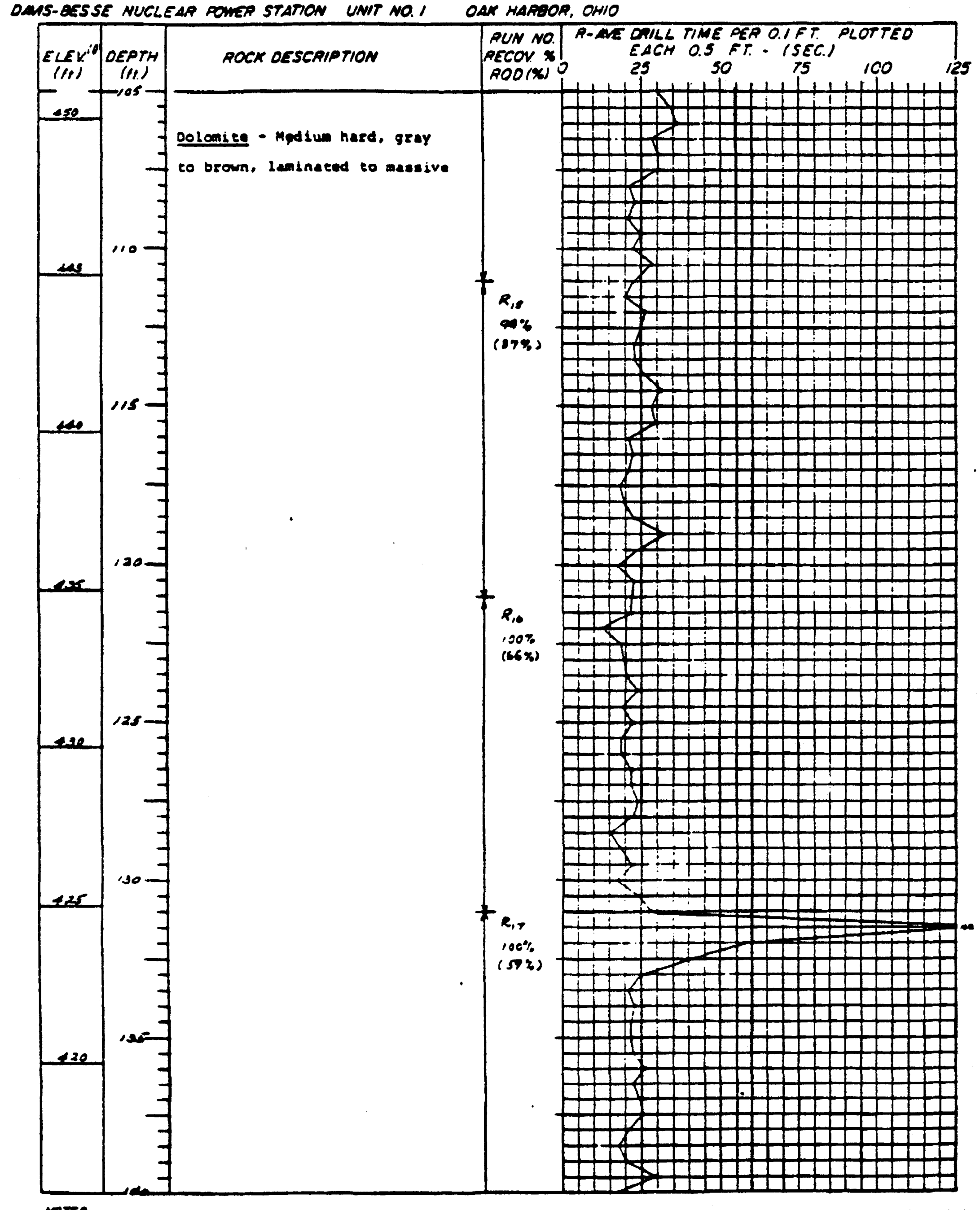
DAVIS-BESSE NUCLEAR POWER STATION UNIT NO. 1 OAK HARBOR, OHIO



NOTES.
(1) ELEVATIONS REFERENCED TO INTERNATIONAL GREAT LAKES DATUM (IGLD)

DESIGN BORING LOG - BORING NO. B7-51(CON'T)
BEARING — ORIENTATION VERTICAL

FIGURE 2C.5-15
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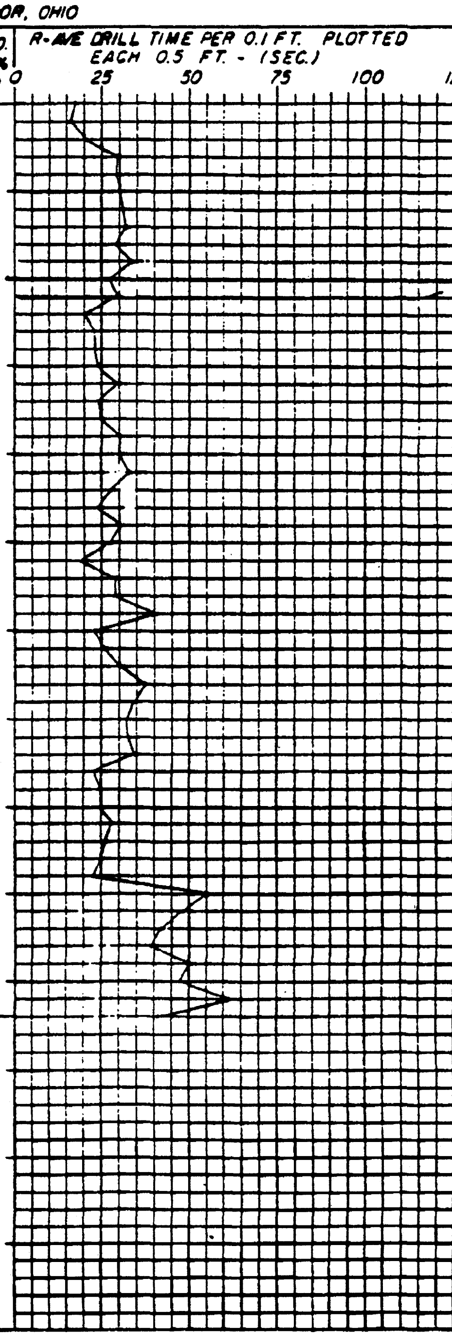
DAVIS-BESSE NUCLEAR POWER STATION UNIT NO. 1 OAK HARBOR, OHIO

ELEV.[1] (ft)	DEPTH (ft.)	ROCK DESCRIPTION	RUN NO. RECOV % RQD(%)
415	140	Dolomite - Medium hard, gray to brown, laminated to massive	R_{18} 88% (63%)
410	145		
405	150		R_{19} 100% (37%)
400	155		
395	160		R_{20} 100% (72%)
390	165	ELEV. BOTTOM OF BORING 389.8	R_{21} 100% (32%)
385	170		
	175		



NOTES.
(1) ELEVATIONS REFERENCED TO INTERNATIONAL GREAT LAKES DATUM (IGLD)

DESIGN BORING LOG - BORING NO. B7-21 (CON'T.)
BEARING — ORIENTATION VERTICAL

FIGURE 2C.5-16

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Ground Surface El. 555.8

Project: Davis-Besse Nuclear Power Plant **Boring** B7-51 **Water Table El.** ______

Core No.	Recovery %	RQD %	Rock type	Top el	Thick-ness ft	Classification and Description
R1	90	26	Dol	555.8	2.8	Medium hard bedded to thinly bedded gray DOLOMITE
						and gypsiferous DOLOMITE with layers of gypsum,
			'			15% banding 1/32 to 3/16 inch thick small bands
						to partings of 1% anhydrite 1/64 to 1/32 inch thick
R2	100	0	Dol	552.7	2.8	Medium hard thinly bedded gray to buff DOLOMITE and
						gypsiferous dolomite with layers of gypsum, 15%
						banding 1/32 to 3/16 inch thick, 1% partial solution
						fissures showing gypsum, selenite
R3	100	41	Dol	549.9	.6	Same as Above
					3.5	Medium hard to hard, irregularly bedded to massive
						ANHYDRITE DOLOMITE with nodules of 2% anhydrite
						mottled in 1/32 to 1/4 inch wide, with 1% occasional
						stringers of gypsum 1/32 to 1/16 inch thick
R4	100	77	Dol	545.8	3.4	Medium hard gray to light gray irregularly bedded
						argillaceous DOLOMITE with pyrite nodules 1/64 inch
						wide in some partings
			Dol		2.0	Medium hard gray thinly bedded DOLOMITE with 2%
						gypsum bands 1/32 to 1/4 inch wide, selenite and
						satin spar, pyrite nodules in some partings, 10%
						banding, 5% partial solution fissures
			Dol		1.1	Medium hard gray to light gray irregularly bedded
						DOLOMITE with gypsum in partings and mottled 1%
						anhydrite nodules 1/16 to 1/8 inch wide
			Dol		1.0	Medium hard gray mottled DOLOMITE with mottled
						gypsum 1/4 to 1 inch wide, mottled anhydrite
						nodules 1/32 to 1/16 inch wide, 20% gypsum
			Dol		1.9	Medium hard, light gray, thinly bedded DOLOMITE
						with gypsum layers 1/32 to 1/16 inch, 15% banding
			Dol		0.6	Medium hard, buff, irregularly laminated DOLOMITE
						with 1% gypsum layers or partings 1/32 to 1/16 inch
						thick and occasional mottled anhydrite nodules
R5	100	43	Dol	535.8	1.0	Same as Above
			Dol		1.4	Soft to medium hard, buff to white, thinly bedded
						DOLOMITE to gypsiferous DOLOMITE with 4% satin spar
						seams 1/16 to 1/8 inch thick
			Dol		1.7	Soft to medium hard, buff, thinly bedded DOLOMITE
						and GYPSIFEROUS DOLOMITE with 5% seams of gypsum
						1/64 to 1/32 inch thick, 1% partial solution fissures
			Gyp/Anh		1.6	Soft to medium hard, buff to brown GYPSUM and
						ANHYDRITE in bands 1/8 to 1 inch
			Dol		2.3	Hard to medium hard, buff to white, thinly bedded
						gypsiferous DOLOMITE with 5% gypsum seams 1/64 to
						1/16 inch thick, 1% partial solution fissures

ROCK CLASSIFICATION DATA SHEET

FIGURE 2C.5-17
REVISION 0
JULY 1982

WMAI 68-192

Ground Surface El. 555.8

Project: Davis-Besse Nuclear Power Plant   Boring B7-51   Water Table El. ______

Core No.	Recovery %	RQD %	Rock type	Top el	Thickness ft	Classification and Description
R6	99	20	Dol	525.8	1.8	Same as Above
			Dol		1.5	Medium hard, buff to gray, thinly bedded argillaceous DOLOMITE and ANHYDROUS DOLOMITE with bands of anhydrite 1/32 to 1/4 inch thick, 10% banding, 3% gypsum and satin spar bands 1/32 to 1/8 inch thick
			Sh		1.4	Medium hard greenish gray, argillaceous DOLOMITE SHALE one satin spar seam 1/8 inch thick
			Dol		2.3	Medium hard, buff to gray, thinly bedded argillaceous DOLOMITE to ANHYDROUS DOLOMITE with 1% bands of anhydrite and gypsum 1/32 to 1/16 inch thick, 1% partial solution fissures
R7	100	49	Dol	516.7	.4	Same as Above
			Dol		4.5	Medium hard, light gray, bedded ANHYDROUS DOLOMITE with one satin spar layer 3/16 inch thick
			Dol/Anh		5.1	Medium hard to hard, light gray, argillaceous ANHYDROUS DOLOMITE to DOLOMITE with 4% satin spar layers 1/16 to 1/8 inch thick and gypsum stringers 1/32 to 1/16 inch thick, 1% partial solution fissures in bottom three feet
R8	100	32	Dol/Anh	506.7	1.9	Medium hard, gray to buff, ANHYDROUS DOLOMITE to DOLOMITE, with gypsum (3%) partings 1/32 inch thick and 1% anhydrite nodules 1/8 inch wide
			Dol		1.6	Medium hard, buff to white, thinly bedded gypsiferous DOLOMITE with bands of anhydrite to 1/16 inch thick, 1% partial solution fissures, 1% anhydrite nodules, 1/32 inch wide
			Sh		4.7	Medium hard gray and green gray, argillaceous to shaley DOLOMITE with one satin spar seam 1/4 inch thick with 1% layers of gypsum and anhydrite (1%) 1/64 to 1/32 inch thick
			Dol/Anh		1.8	Medium hard to hard, brown and buff, DOLOMITE and ANHYDRITE bands 1/8 to 1/2 inch thick with gypsum bands 1/16 inch thick, 3% partial solution fissures
R9	100	24	Dol/Anh	496.7	1.8	Medium hard, buff, thinly bedded argillaceous shaley DOLOMITE, gypsiferous DOLOMITE and ANHYDRITE, bands 1/32 to 1/8 inch thick
			Dol		.8	Medium hard, gray, bedded DOLOMITE with mottled anhydrite stringers and pyrite nodules
			Anh		2.3	Medium hard, brown, thinly bedded to shaley ANHYDRITE
R10	100	77	Anh	491.8	1.7	Same as Above
			Anh		4.9	Medium hard to hard, brown massive DOLOMITIC ANHYDRITE
			Dol/Anh		.4	Medium hard, light gray and brown, mottled to irregularly bedded DOLOMITE and ANHYDRITE with anhydrite nodules 1/16 to 1/4 inch wide
			Dol		2.0	Medium hard, buff, massive DOLOMITE, with anhydrite nodules 1/32 to 1/8 inch wide

ROCK CLASSIFICATION DATA SHEET

FIGURE 2C.5-18

REVISION 0
JULY 1982

WMAI 68-192

Ground Surface El. 555.8

Project: Davis-Besse Nuclear Power Plant Boring B7-51 Water Table El. ______

Core No.	Recovery %	RQD %	Rock type	Top el	Thickness ft	Classification and Description
			Dol/Anh		.6	Medium hard to hard, mottled DOLOMITE and ANHYDRITE with mottled gypsum 1/8 inch wide, pyrite nodules seen in partings
			Anh		.2	Medium hard, brown, massive ANHYDRITE to DOLOMITIC ANHYDRITE
R11	97	62	Anh	482.0	7.0	Same as Above
R12	100	64	Anh	474.8	1.9	Same as Above
			Dol		2.1	Medium hard, buff, irregularly to thinly bedded DOLOMITE with 1/4 inch seam of satin spar and mottled stringers and 3% nodules of anhydrite
			Anh		2.8	Medium hard to hard, massive brown to gray, ANHYDRITE with 1% gypsum 1/16 to 1/4 inch mottles
			Anh		.3	Medium hard, buff to brown, thinly bedded ANHYDRITE and 1% gypsum layers 1/16 to 1/8 inch thick
			Dol		2.9	Medium hard, buff, thinly bedded to irregularly bedded DOLOMITE with 1% gypsum, satin spar and anhydrite from 1/32 to 1/4 inch thick, 30% banding, 3% partial solution fissures
R13	100	60	Dol	464.8	1.5	Same as Above
			Dol		5.5	Medium hard, gray, irregularly bedded DOLOMITE with black shaley partings and stringers 1/64 inch wide and occasional 1% nodules of anhydrite 1/32 to 1/64 inch wide
			Anh		3.0	Medium hard irregularly bedded ANHYDRITE to DOLOMITIC ANHYDRITE with 1% satin spar seams 1/32 to 1/4 inch thick with .1 foot thinly bedded dolomite
R14	100	93	Anh	454.8	2.5	Same as Above
			Dol		1.8	Medium hard, buff, thin to medium bedded, DOLOMITE with mottled gypsum 1 inch wide and satin spar seams 1/32 to 1/8 inch wide and anhydrite nodules 1/64 to 1/32 inch wide and black shaley stringers 1/64 inch wide
			Dol		1.7	Medium hard, buff, irregularly bedded DOLOMITE with 3% 1/8 to 1/4 inch gypsum pockets and black shaley stringers and partings 1/64 inch wide
R15	99	87	Dol	444.8	9.9	Same as Above
R16	100	66	Dol	434.8	10.0	Same as Above
R17	100	57	Dol	424.8	7.4	Medium hard, buff to gray, massive to irregularly bedded DOLOMITE with black shaley stringers and partings and one gypsum seam 1/8 inch wide
			Dol		2.6	Medium hard, buff, massive DOLOMITE
R18	88	63	Dol	414.8	7.9	Same as Above
R19	100	27	Dol	405.8	8.5	Medium hard, light gray, massive DOLOMITE with occasional sections of thin bedding with 1% black shaley stringers and partings 1/64 inch wide
R20	100	71	Dol	397.3	4.2	Same as Above

ROCK CLASSIFICATION DATA SHEET

FIGURE 2C.5-19

WMAI 68-192

REVISION 0
JULY 1982

Ground Surface El. 555.8

Project: Davis-Besse Nuclear Power Plant   Boring B7-51   Water Table El. ____

Core No.	Recovery %	RQD %	Rock type	Top el	Thick-ness ft	Classification and Description
R21	100	82	Anh	193.1	2.5	Medium hard, massive, brown ANHYDRITE with 1%
						occasional mottled gypsum or gypsiferous anhydrite
			Dol		.8	Medium hard, gray, massive DOLOMITE with 1% black
						stringers and partings 1/64 inch wide
				189.8		BOTTOM ELEVATION

ROCK CLASSIFICATION DATA SHEET

FIGURE 2C.5-20

REVISION 0
JULY 1982

WMAI 68-192

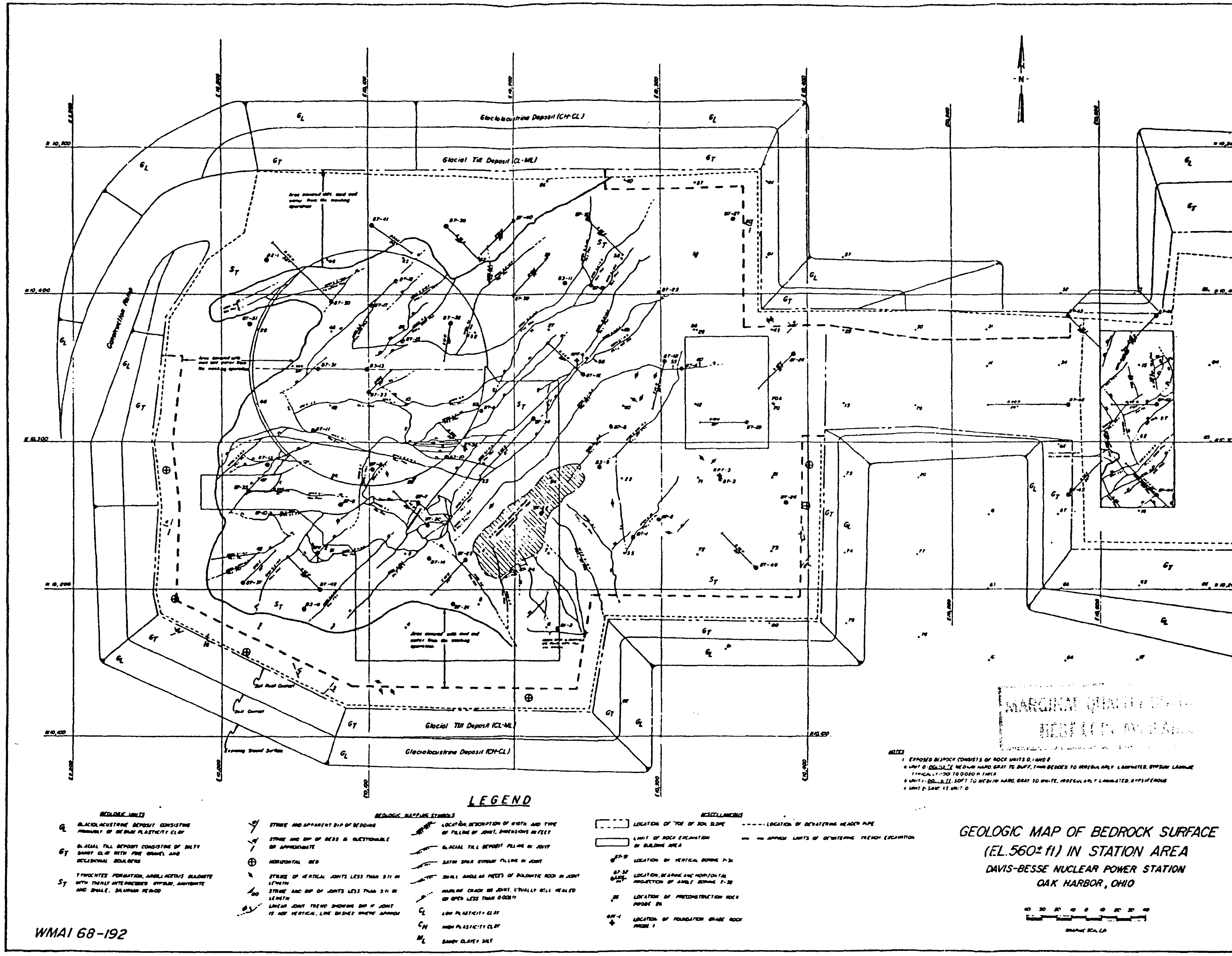


FIGURE 2C.5-21 REVISION 0
JULY 1982

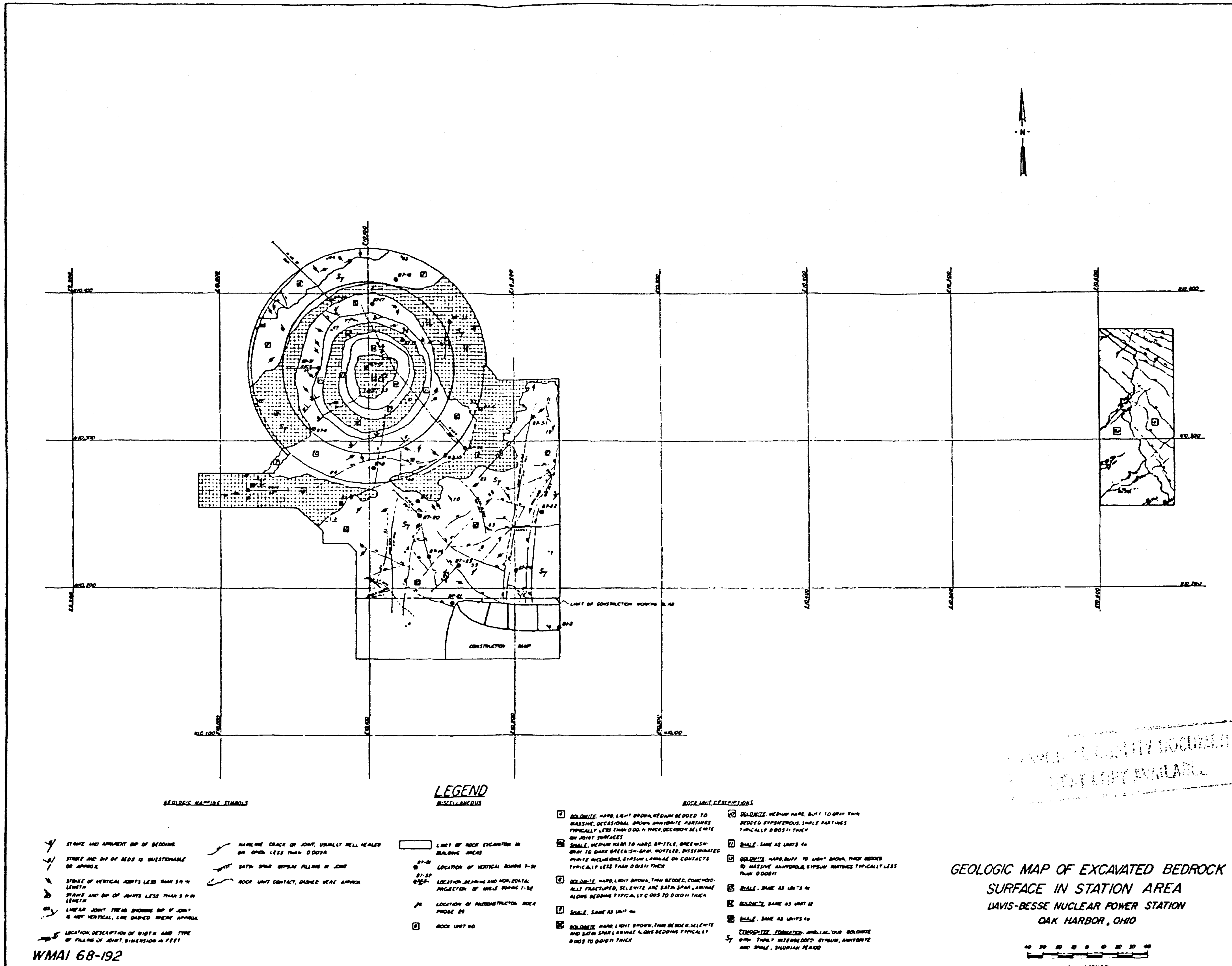
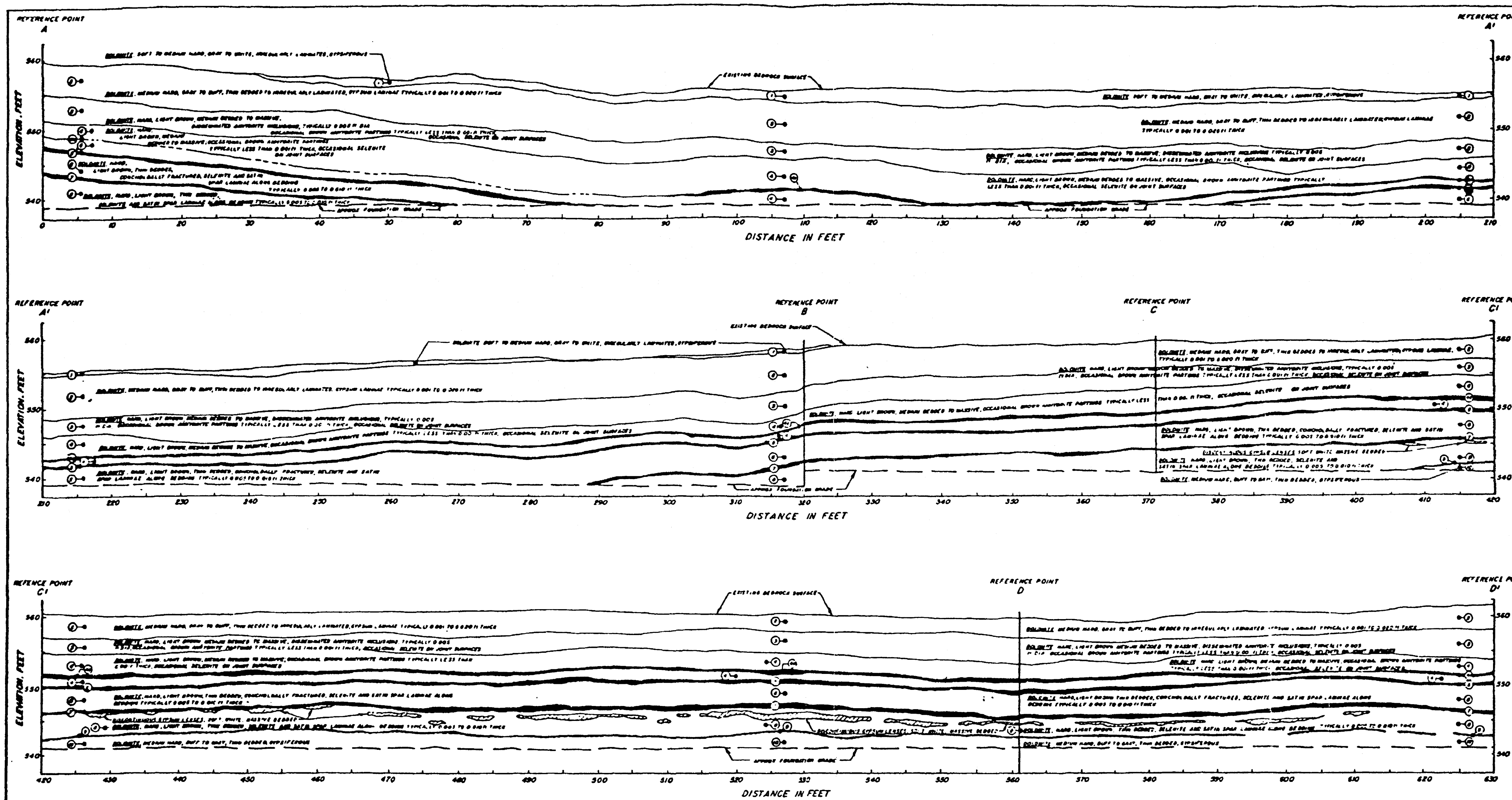
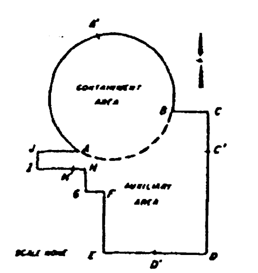


FIGURE 2C.5-22 REVISION 0
JULY 1982





REFERENCE POINT LOCATION SKETCH

NOTES:

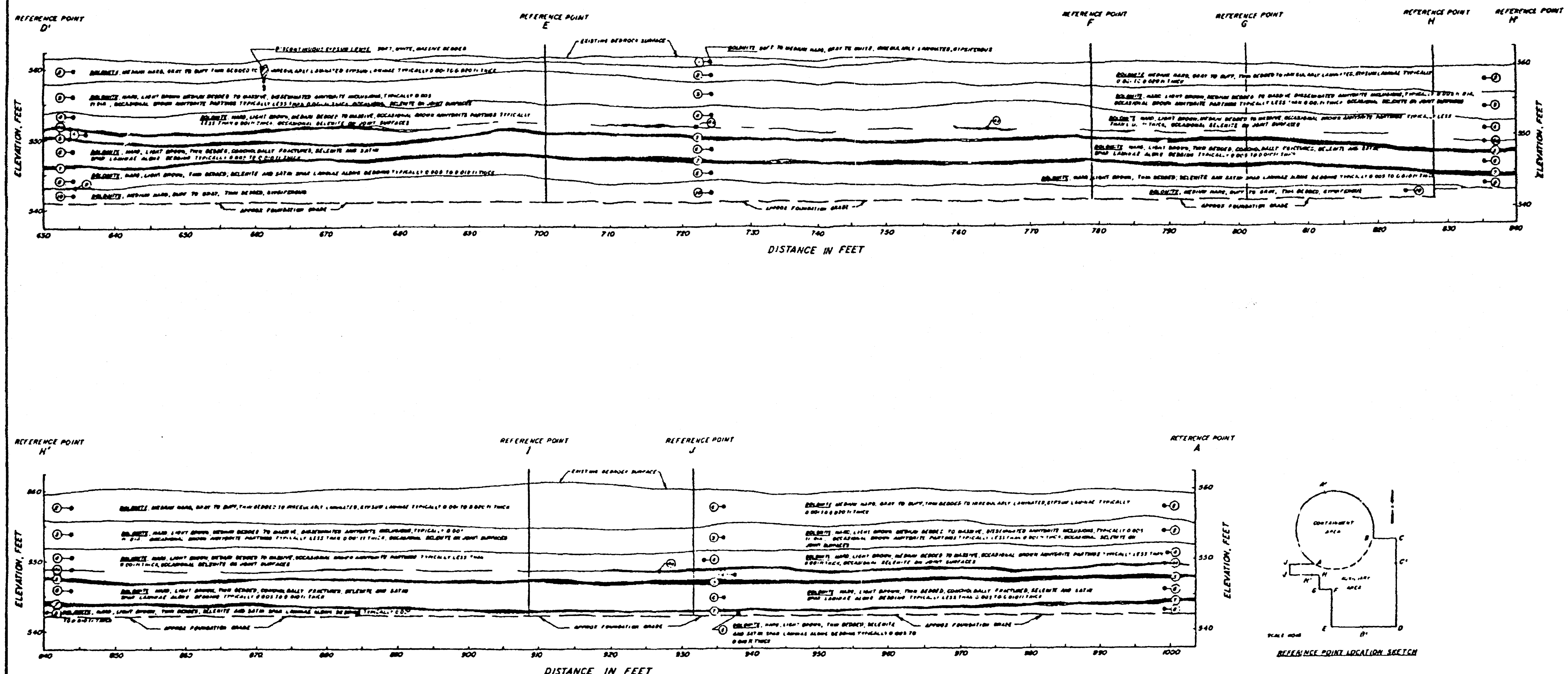
1 ELEVATIONS REFERENCED TO INTERNATIONAL GREAT LAKES DATUM (IGLD)
2 EXPOSED ROCK - SILURIAN PERIOD, TYMOCHTEE FORMATION
3 BEDDING IN MAPPED AREA IS GENERALLY HORIZONTAL, EXCEPT IN THE CONTAINMENT AREA WHERE BEDDING DIPS APPROX 5° NW.
4 FOR LOCATION OF REFERENCE POINTS, SEE LOCATION SKETCH ON THIS FIGURE.
5 DISTANCE IN FEET MEASURED AS LINEAL DISTANCE ALONG PRESPLIT WALL FROM REFERENCE POINT A
6 NUMBERS IN CIRCLES REFER TO ROCK UNIT DESIGNATION
7 DESCRIPTION OF ROCK UNITS 2a, 5, 7 AND 9 ARE NOT INCLUDED ON SECTIONS THESE UNITS CONSIST OF SHALE AND DISCONTINUOUS SHALE BEDS AND ARE DESCRIBED AS FOLLOWS SHALE. MEDIUM HARD TO HARD, BRITTLE, GREENISH GRAY TO DARK GREENISH GRAY, MOTTLED, DISSEMINATED PYRITE INCLUSIONS, GYPSUM LAMINAE ON CONTACTS TYPICALLY LESS THAN 0.018 ft THICK.

GEOLOGIC SECTIONS
CONTAINMENT AND AUXILIARY BUILDING EXCAVATION WALLS
DAVIS-BESSE NUCLEAR POWER STATION
OAK HARBOR, OHIO

10 5 0 5 10
GRAPHIC SCALE, ft

WMAI 68-192

FIGURE 2C.5-23 REVISION 0
JULY 1982



NOTES:

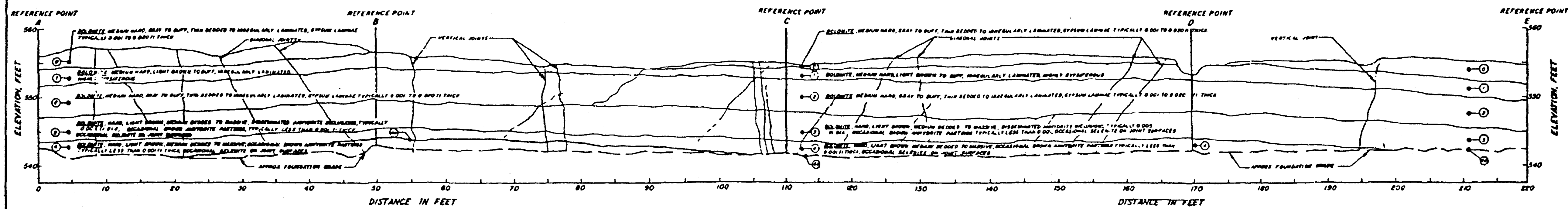
1 ELEVATIONS REFERENCED TO INTERNATIONAL GREAT LAKES DATUM (IGLD)
2 EXPOSED ROCK SILURIAN PERIOD, TYMOCHTEE FORMATION
3 BEDDING IN MAPPED AREA IS GENERALLY HORIZONTAL, EXCEPT IN THE CONTAINMENT AREA WHERE BEDDING DIPS APPROX 5° NW.
4 FOR LOCATION OF REFERENCE POINTS, SEE LOCATION SKETCH ON THIS FIGURE.
5 DISTANCE IN FEET MEASURED AS LINEAL DISTANCE ALONG PRESPLIT WALL FROM REFERENCE POINT A.
6 NUMBERS IN CIRCLES REFER TO ROCK UNIT DESIGNATION
7 DESCRIPTION OF ROCK UNITS 4, 5, 7 AND 9 ARE NOT INCLUDED ON SECTIONS. THESE UNITS CONSIST OF SHALE TO DISCONTINUOUS SHALE BEDS AND ARE DESCRIBED AS FOLLOWS: SHALE, MEDIUM HARD TO HARD, BRITTLE, GREENISH GRAY TO DARK GREENISH GRAY, MOTTLED, DISSEMINATED PYRITE INCLUSIONS, GYPSUM LAMINAE ON CONTACTS TYPICALLY LESS THAN 0.015 FT THICK

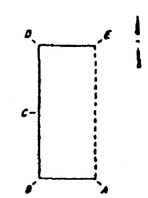
GEOLOGIC SECTIONS
CONTAINMENT AND AUXILIARY BUILDING EXCAVATION WALLS
DAVIS-BESSE NUCLEAR POWER STATION
OAK HARBOR, OHIO

10 5 0 5 10
GRAPHIC SCALE, ft

WMAI 68-192

FIGURE 2C.5-24 REVISION 0
JULY 1982





INTAKE AREA

SCALE NONE

INTAKE AREA

REFERENCE POINT LOCATION SKETCH

NOTES

1 ELEVATIONS REFERENCED TO INTERNATIONAL GREAT LAKES DATUM (IGLD)

2 EXPOSED ROCK SILURIAN PERIOD, TYMOCHTEE FORMATION.

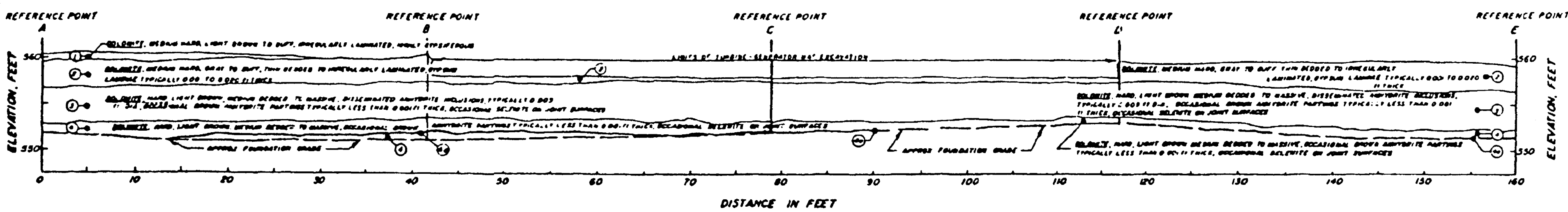
3 FOR LOCATION OF REFERENCE POINTS, SEE LOCATION SKETCHES ON THIS FIGURE.

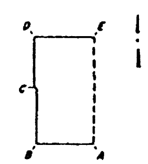
4 DISTANCE IN FEET MEASURED AS LINEAL DISTANCE ALONG PRESPLIT WALL FROM REFERENCE POINT A

5 NUMBERS IN CIRCLES REFER TO ROCK UNIT DESIGNATION

6 DESCRIPTION OF ROCK UNIT [illegible] IS NOT INCLUDED ON SECTIONS THIS UNIT CONSISTS OF SHALE TO DISCONTINUOUS SHALE BEDS AND IS DESCRIBED AS FOLLOWS: SHALE. MEDIUM HARD TO HARD, BRITTLE, GREENISH GRAY TO DARK GREENISH GRAY, MOTTLED, DISSEMINATED PYRITE INCLUSIONS, GYPSUM LAMINAE ON CONTACTS TYPICALLY LESS THAN 0.015 ft THICK

7 VERTICAL JOINTS, TYPICALLY HAIRLINE TO 0.008 ft, GYPSUM FILLED DIAGONAL JOINTS, TYPICALLY HAIRLINE USUALLY WELL HEALED OR LESS THAN 0.003 ft





CIRCULATING WATER DUCT AREA

SCALE NONE

CIRCULATING WATER DUCT AREA

REFERENCE POINT LOCATION SKETCH

GEOLOGIC SECTIONS

INTAKE AND CIRCULATING WATER DUCT EXCAVATION WALLS

DAVIS-BESSE NUCLEAR POWER STATION

OAK HARBOR, OHIO

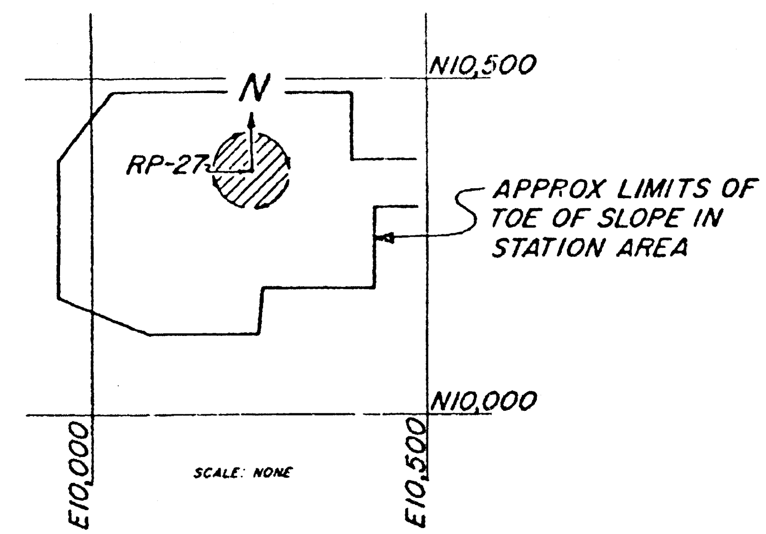
GRAPHIC SCALE, ft

WMAI 68-192

FIGURE 2C.5-25 REVISION 0
JULY 1982

NOTES:

1. RP-27 REFERS TO PRECONSTRUCTION ROCK PROBE NUMBER 27. (N10375, E10224)
2. APPROXIMATE LIMITS OF AREA INCLUDED IN THE PHOTOGRAPH INDICATED AS CROSS HATCHED AREA IN THE PHOTOGRAPH REFERENCE SKETCH.
3. SIMILAR PHOTOGRAPHS TAKEN OF: (A) BEDROCK SURFACE AT EL 560±, (B) EXCAVATED BEDROCK SURFACES IN THE MAT PORTION OF THE AUXILIARY BUILDING AND THE INTAKE AREAS AND (C) VERTICAL EXCAVATION WALLS IN ALL EXCAVATIONS MORE THAN 5ft DEEP (PIER EXCAVATIONS NOT PHOTOGRAPHED).



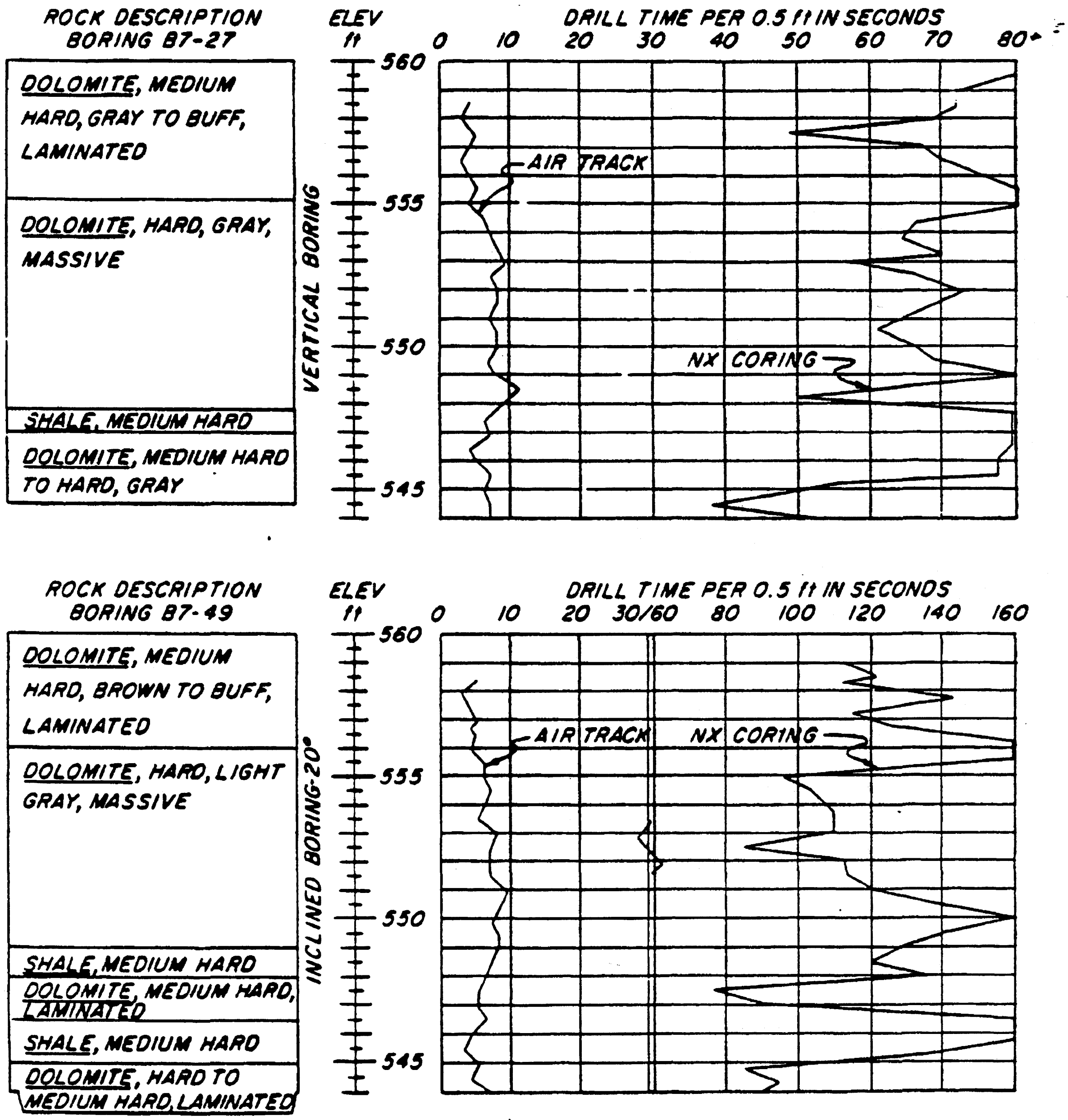
PHOTOGRAPH REFERENCE SKETCH

TYPICAL ROCK SURFACE PHOTOGRAPH-TAKEN AT RP-27 IN THE NORTH AUXILIARY BUILDING AREA-EL 560±

WMAI 68-192

FIGURE 2C.5-26

REVISION 0
JULY 1982

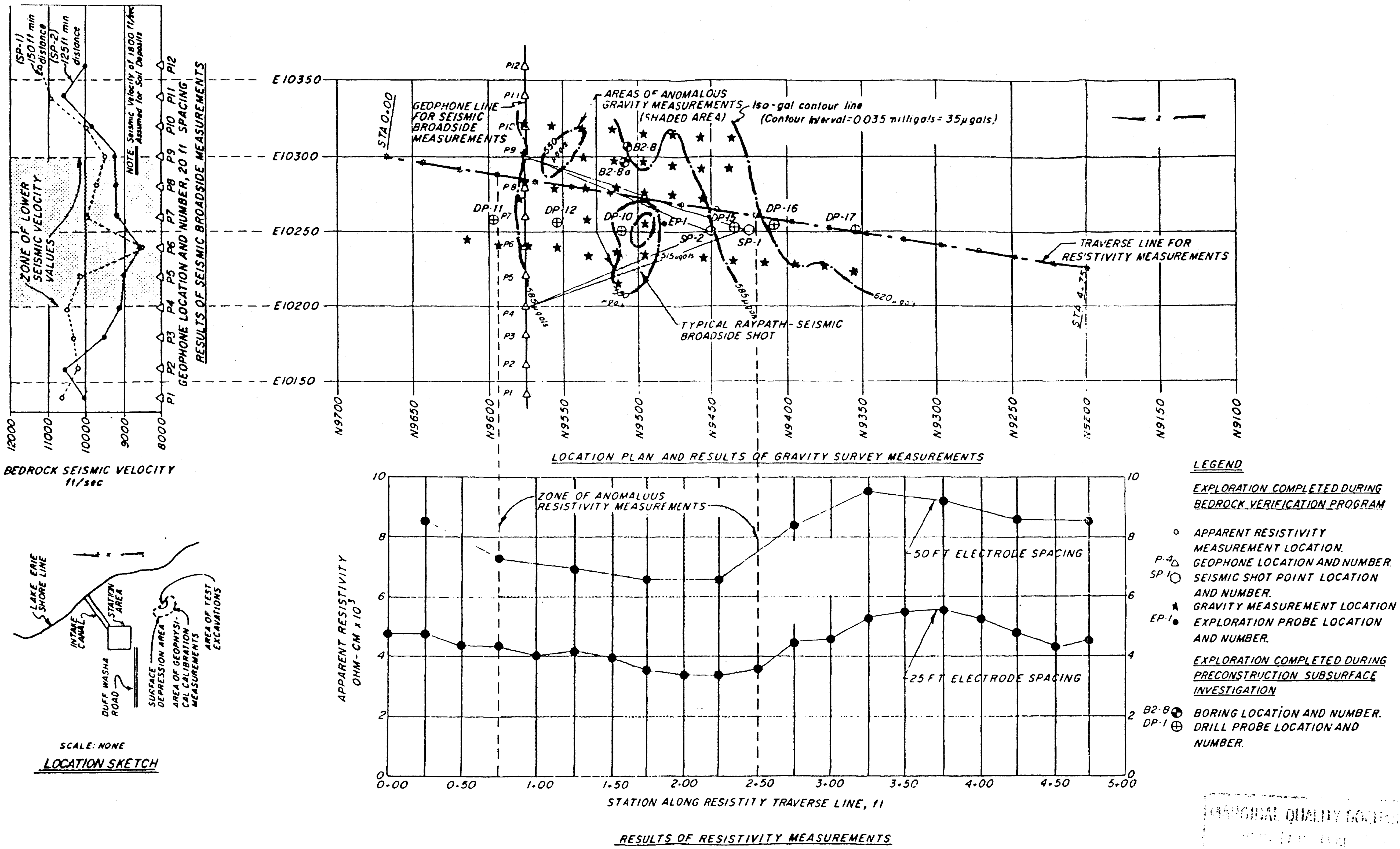


NOTES:
1. ROCK PROBE AT PIER MARK A_b-2 WAS MADE 9 ft FROM B7-27.
2. ROCK PROBE AT PIER MARK A_a-10 WAS MADE 13.5 ft FROM B7-49.
3. NO SIGNIFICANT SOLUTION ACTIVITY WAS ENCOUNTERED IN EITHER BORING.

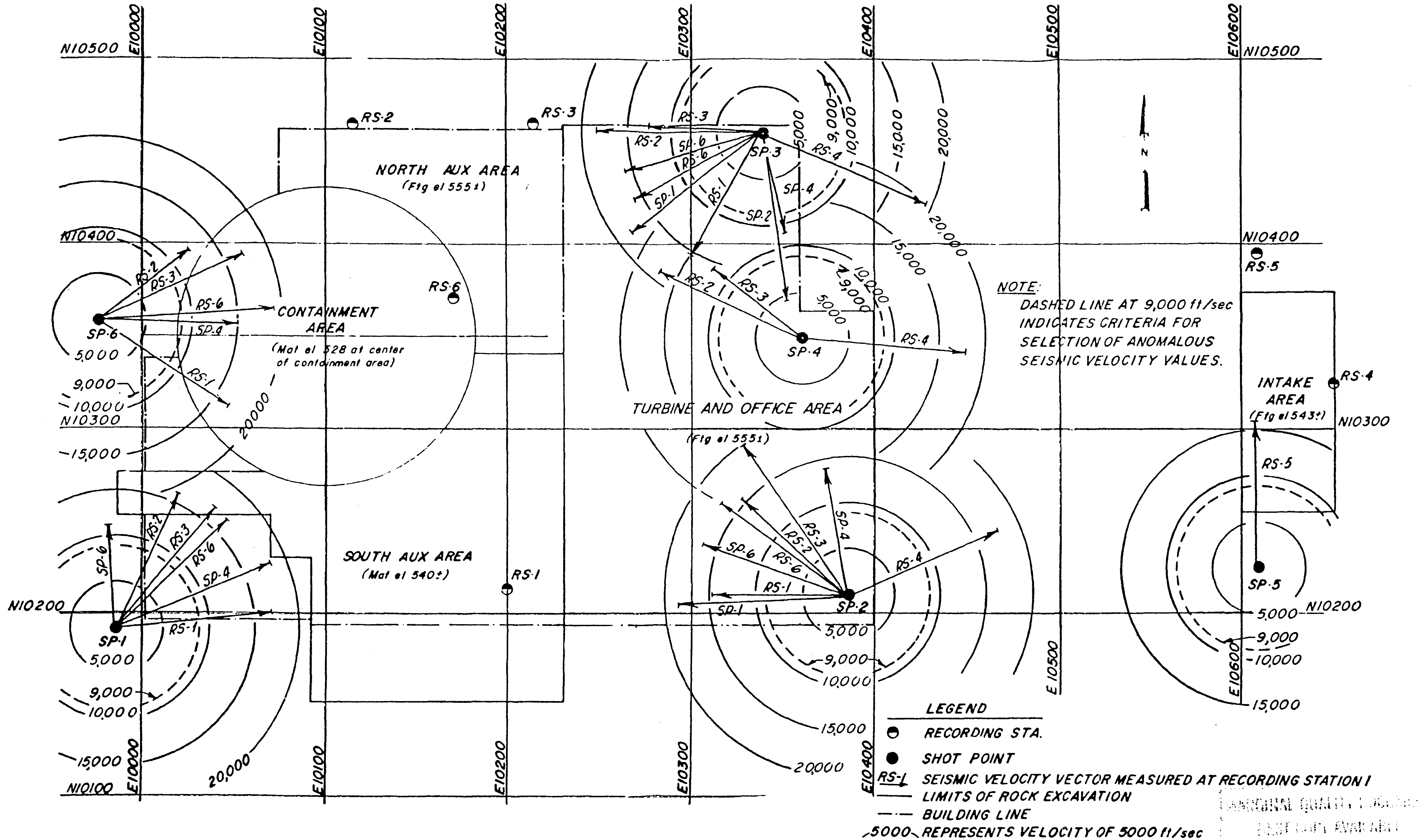
COMPARISON OF AIR TRACK AND NX CORE DRILL TIMES

WMAI 68-192

FIGURE 2C.5-27
REVISION 0
JULY 1982



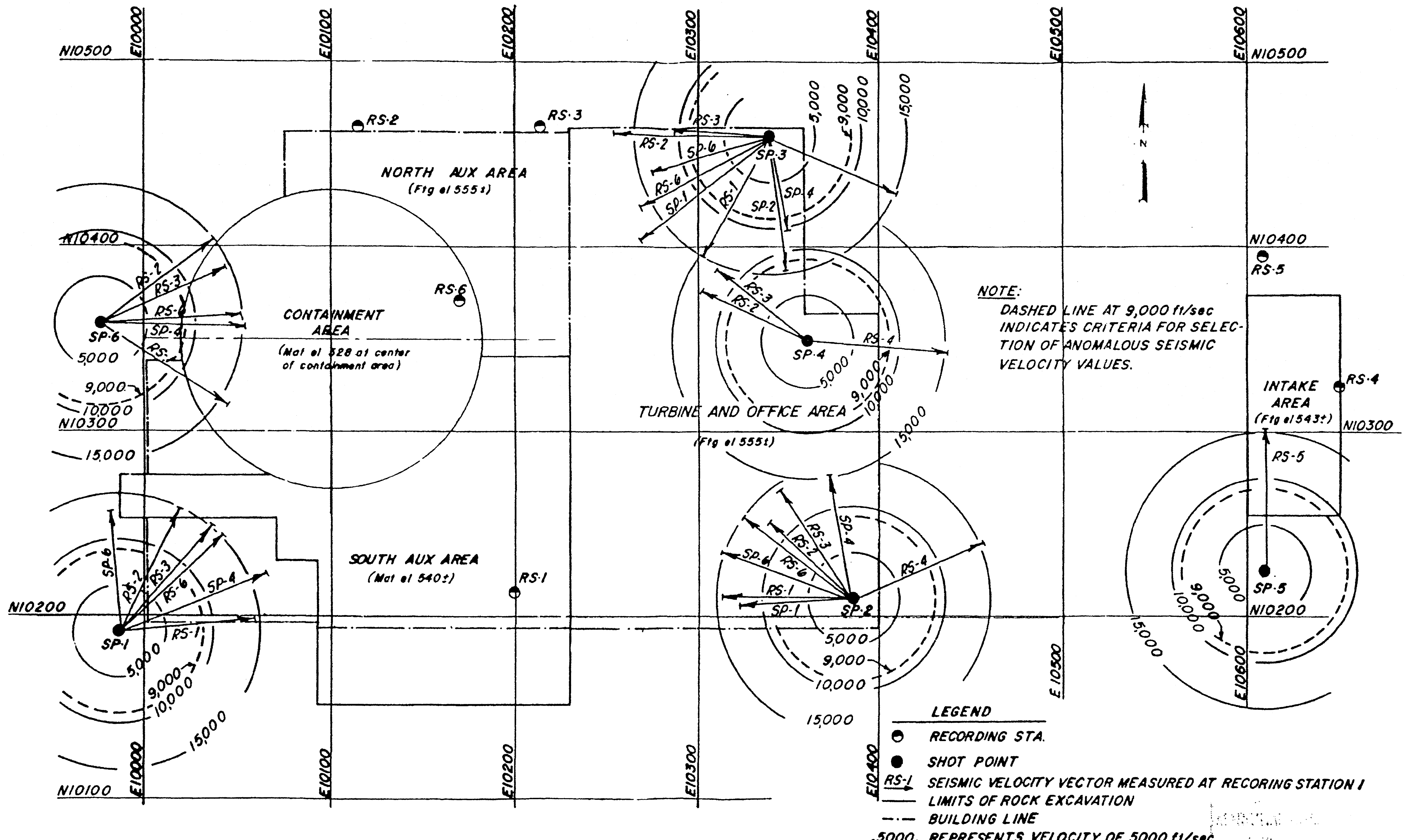
RESULTS OF GEOPHYSICAL CALIBRATION MEASUREMENTS MADE IN THE SURFACE DEPRESSION AREA



SEISMIC VELOCITIES MEASURED AT ELEVATION 490

WMAI 68-192

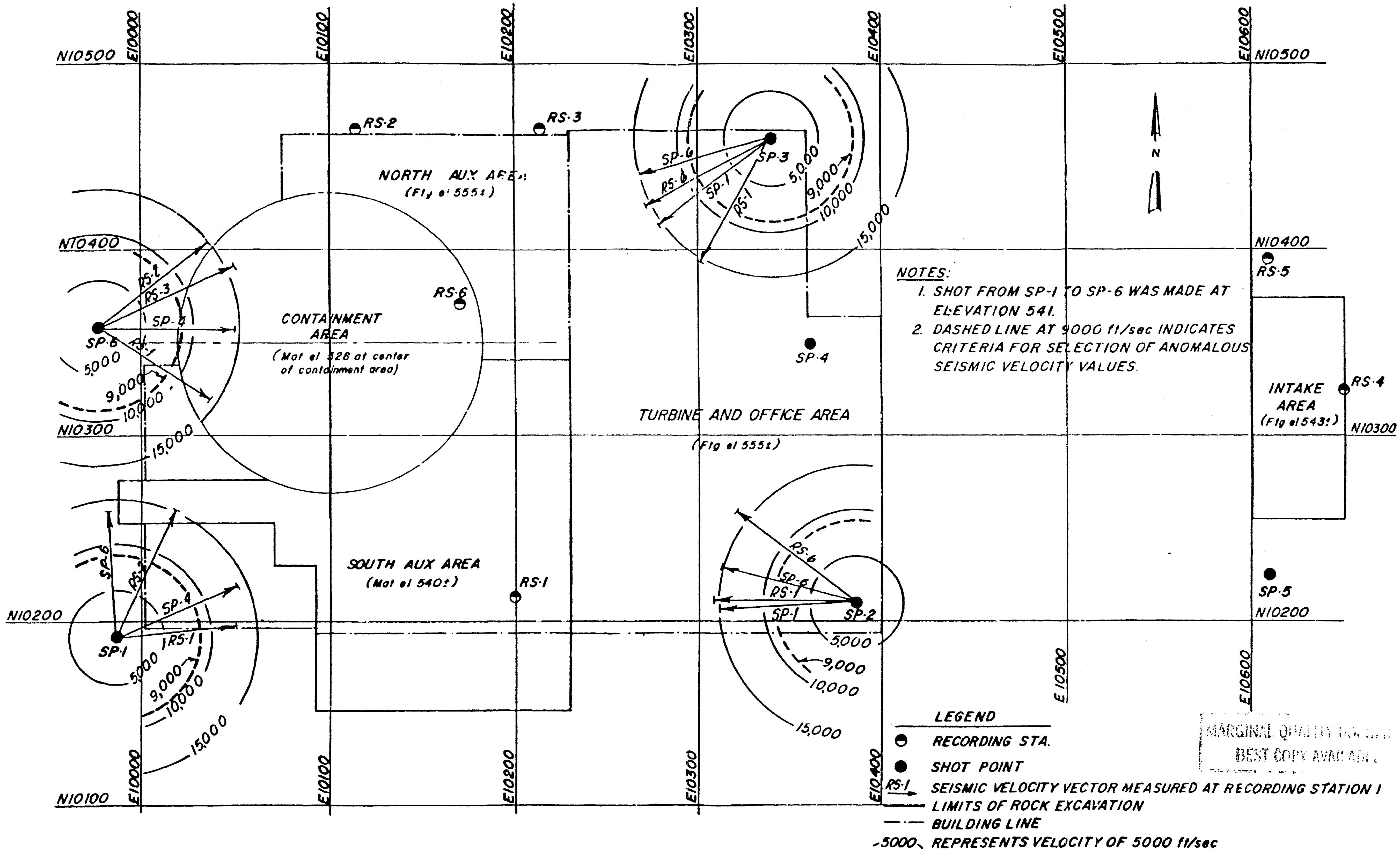
FIGURE 2C.5-29
REVISION 0
JULY 1982



SEISMIC VELOCITIES MEASURED AT ELEVATION 520

MAI 68-192

FIGURE 2C.5-30
REVISION 0
JULY 1982

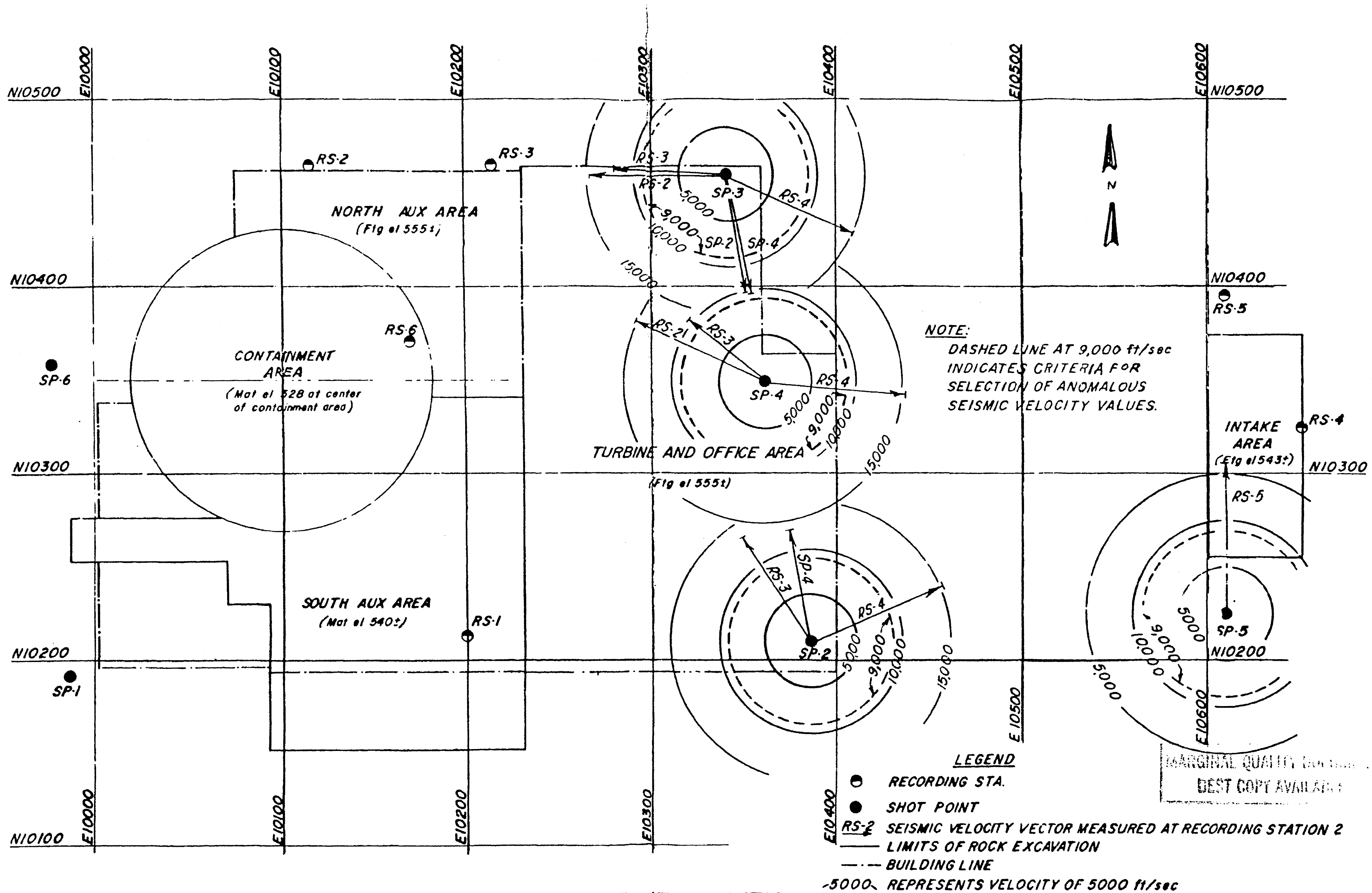


MARGINAL QUALITY DOCUMENT BEST COPY AVAILABLE

SEISMIC VELOCITIES MEASURED AT ELEVATION 535

WMAI 68-192

FIGURE 2C.5-31
REVISION 0
JULY 1982



SEISMIC VELOCITIES MEASURED AT ELEVATION 550

YMAI 68-192

FIGURE 2C.5-32
REVISION 0
JULY 1982

WMAI 68-192

VARIATION IN RELATIVE BOUGUER GRAVITY ACROSS PIER EXCAVATION B_a-1

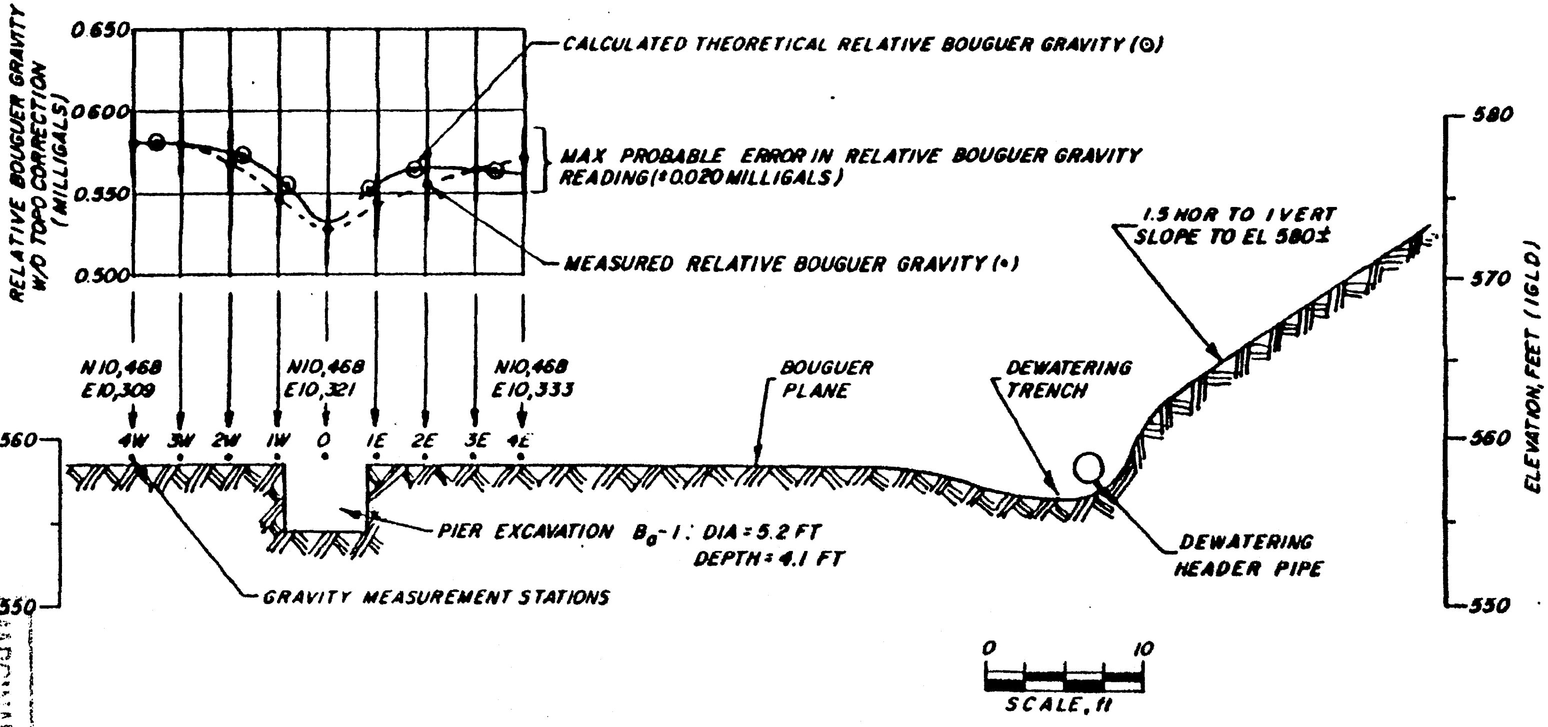


FIGURE 2C.5-33
REVISION 0
JULY 1982

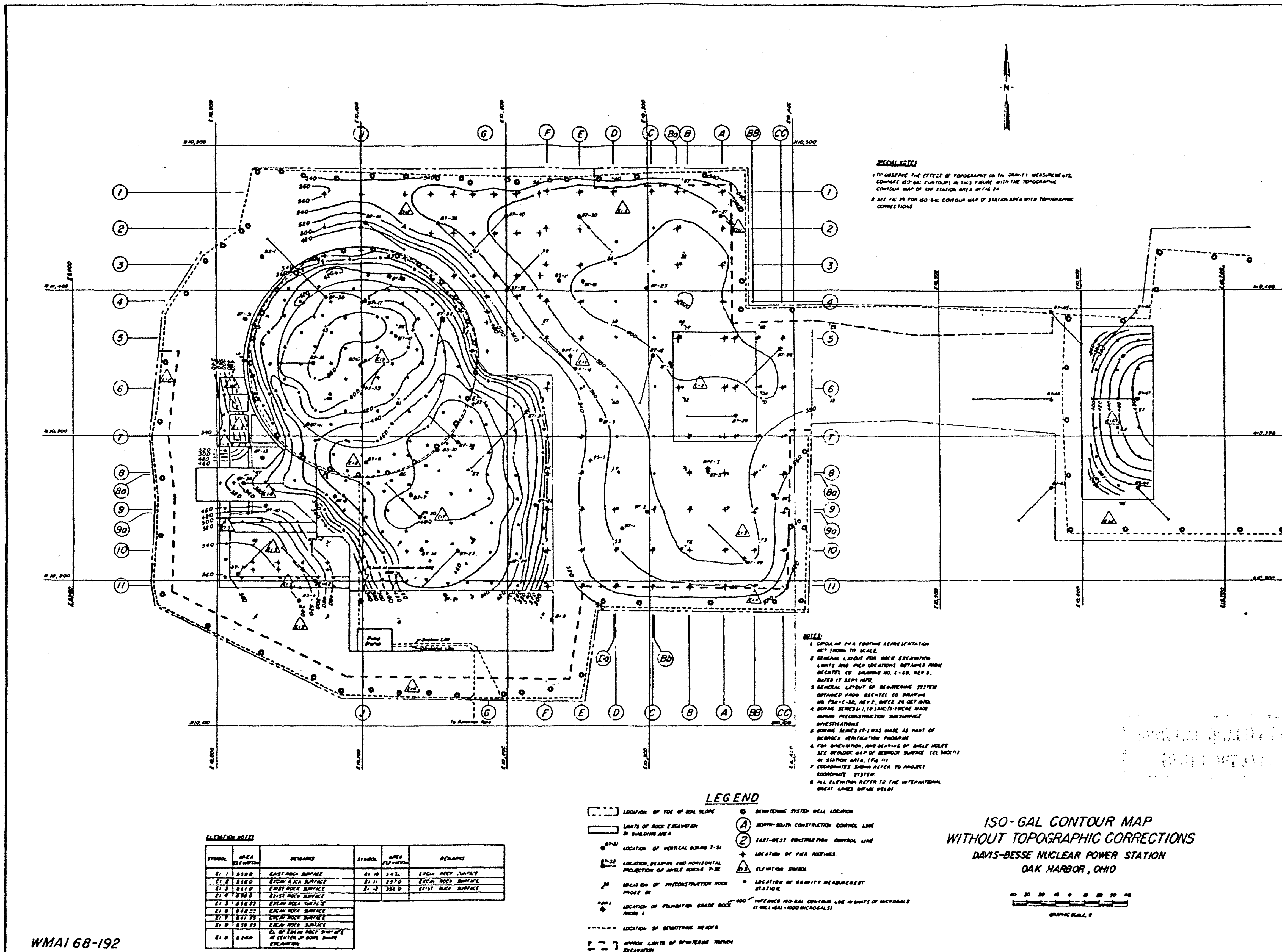
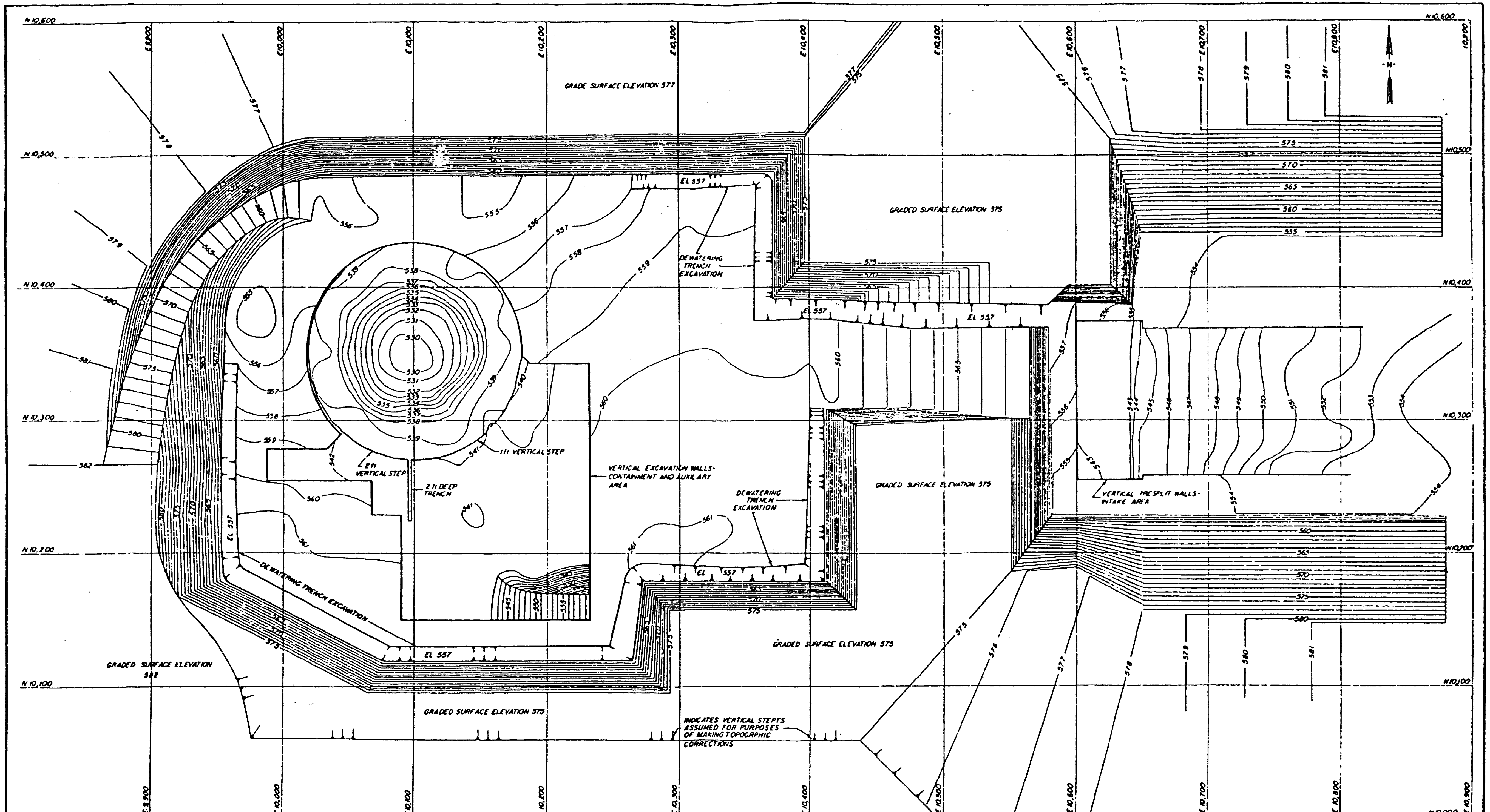


FIGURE 2C.5-34 REVISION 0
JULY 1982



NOTES

1. GENERAL LAYOUT OBTAINED FROM BECHTEL DRAWING NO. C-9, DATEC 8-25-70
2. TOPOGRAPHIC CONTOURS OF SOIL SLOPES ABOVE APPROX EL 560 DETERMINED FROM BECHTEL DRAWING NO. C-9, DATED 8-25-70 TOPOGRAPHIC CONTOURS OF EXCAVATED BEDROCK SURFACES DETERMINED BY OPTICAL SURVEY MEASUREMENTS MADE FOR GRAVITY SURVEY MEASURMENTS
3. ELEVATIONS REFERENCED TO INTERNATIONAL GREAT LAKES DATUM (IGLD).

WMA1 68-192

STATION AREA TOPOGRAPHIC MAP
PREPARED TO MAKE TOPOGRAPHIC CORRECTIONS FOR GRAVITY SURVEY
DAVIS-BESSE NUCLEAR POWER STATION
OAK HARBOR, OHIO

GRAPHIC SCALE, ft

FIGURE 2C.5-35 REVISION 0
JULY 1982

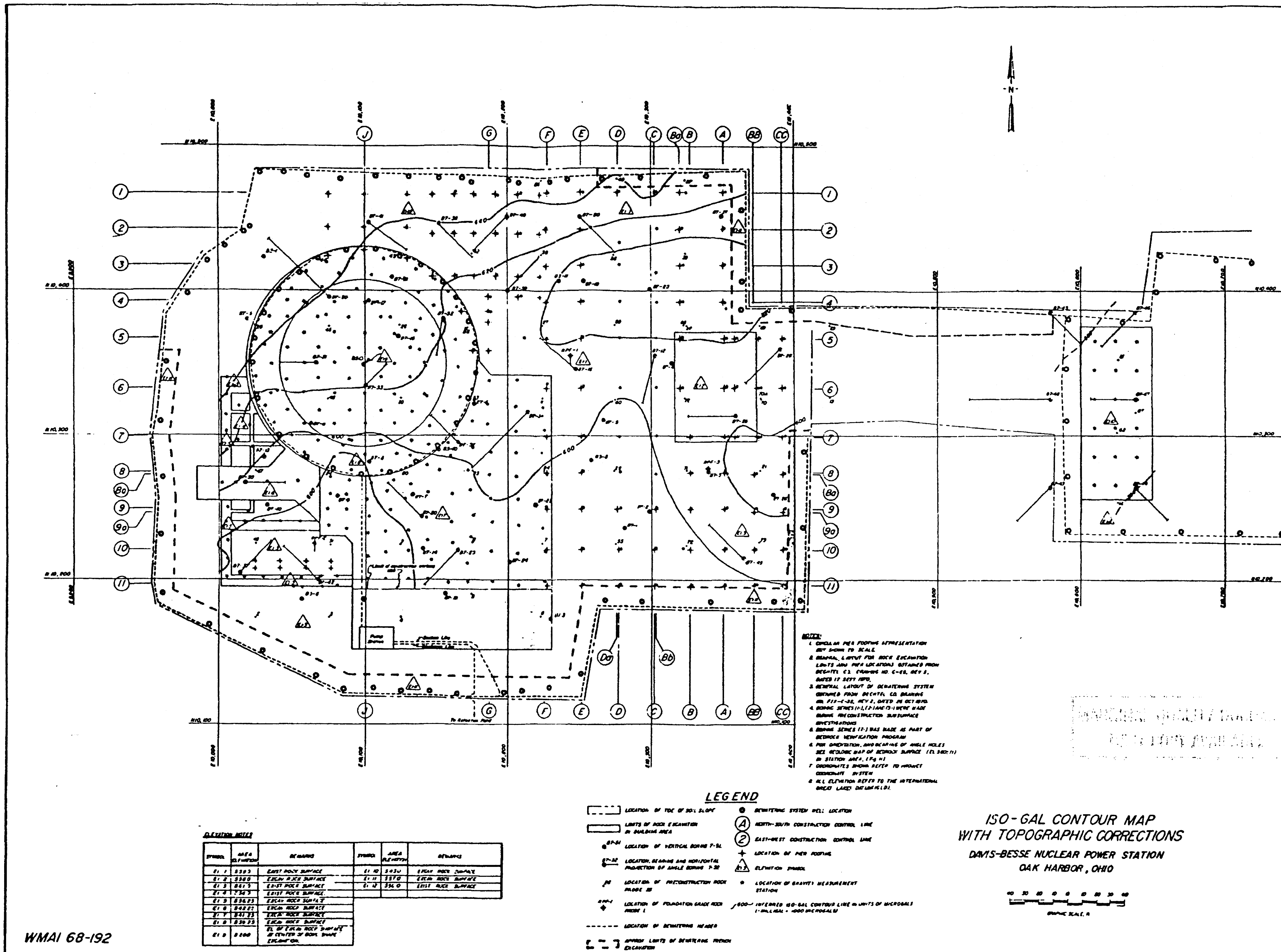
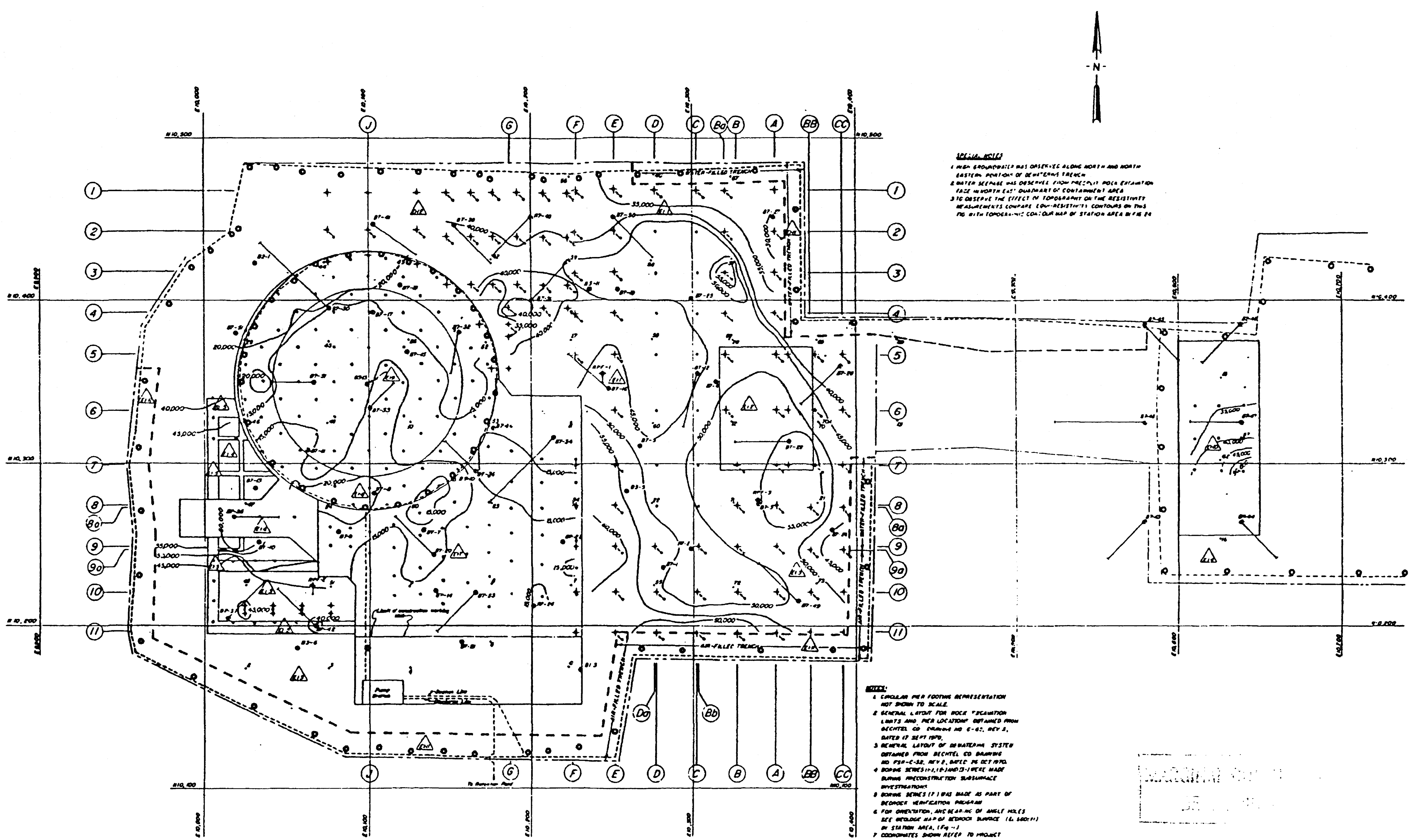


FIGURE 2C.5-36 REVISION 0
JULY 1982



SPECIAL NOTES

1. HIGH GROUNDWATER WAS OBSERVED ALONG NORTH AND NORTH EASTERN PORTION OF DEWATERING TRENCH
2. WATER SEEPAGE WAS OBSERVED FROM PRESPLIT ROCK EXCAVATION FACE IN NORTH EAST QUADRANT OF CONTAINMENT AREA
3. TO OBSERVE THE EFFECT OF TOPOGRAPHY ON THE RESISTIVITY MEASUREMENTS COMPARE EQUI-RESISTIVITY CONTOURS ON THIS FIG WITH TOPOGRAPHIC CONTOUR MAP OF STATION AREA IN FIG 2C

NOTES:

1. CIRCULAR PIER FOOTING REPRESENTATION NOT SHOWN TO SCALE.
2. GENERAL LAYOUT FOR ROCK EXCAVATION LIMITS AND PIER LOCATIONS OBTAINED FROM BECHTEL CO DRAWING NO C-62, REV 5, DATED 17 SEPT 1970.
3. GENERAL LAYOUT OF DEWATERING SYSTEM OBTAINED FROM BECHTEL CO DRAWING NO FSH-C-32, REV 2, DATED 26 OCT 1970.
4. BORING SERIES (1-1), (P-) AND (3-) WERE MADE DURING PRECONSTRUCTION SUBSURFACE INVESTIGATIONS
5. BORING SERIES (7) WAS MADE AS PART OF BEDROCK VERIFICATION PROGRAM
6. FOR ORIENTATION, AND BEARING OF ANGLE HOLES SEE GEOLOGIC MAP OF BEDROCK SURFACE (EL 560±) IN STATION AREA. (Fig --)
7. COORDINATES SHOWN REFER TO PROJECT COORDINATE SYSTEM
8. ALL ELEVATION REFER TO THE INTERNATIONAL GREAT LAKES DATUM (IGLD)

ELEVATION NOTES

SYMBOL	AREA ELEVATION	REMARKS	SYMBOL	AREA ELEVATION	REMARKS
El 1	559.0	EXIST ROCK SURFACE	EL 10	543.0	EXCAV ROCK SURFACE
El 2	556.0	EXCAV ROCK SURFACE	EL 11	557.0	EXCAV ROCK SURFACE
El 3	561.0	EXIST ROCK SURFACE	EL 12	556.0	EXIST ROCK SURFACE
El 4	558.0	EXIST ROCK SURFACE			
El 5	556.25	EXCAV ROCK SURFACE			
El 6	542.25	EXCAV ROCK SURFACE			
El 7	541.75	EXCAV ROCK SURFACE			
El 8	539.25	EXCAV ROCK SURFACE			
El 9	528.0	EL OF EXCAV ROCK SURFACE AT CENTER OF BOWL SHAPE EXCAVATION			

LEGEND

- LOCATION OF TOE OF SOIL SLOPE
- LIMITS OF ROCK EXCAVATION IN BUILDING AREA
- BT-31 LOCATION OF VERTICAL BORING T-31
- BT-32 LOCATION, BEARING AND HORIZONTAL PROJECTION OF ANGLE BORING T-32
- 20 LOCATION OF PRECONSTRUCTION ROCK PROBE 20
- RPF-1 LOCATION OF FOUNDATION GRADE ROCK PROBE 1
- LOCATION OF DEWATERING HEADER
- APPROX LIMITS OF DEWATERING TRENCH EXCAVATION
- DEWATERING SYSTEM WELL LOCATION
- (A) NORTH-SOUTH CONSTRUCTION CONTROL LINE
- (2) EAST-WEST CONSTRUCTION CONTROL LINE
- LOCATION OF PIER FOOTING
- EL 5 ELEVATION SYMBOL
- LOCATION OF RESISTIVITY READING
- LOCATION OF RESISTIVITY READINGS AT PIER FOOTING
- 10,000 INFERRED EQUI-RESISTIVITY CONTOUR IN UNITS OF OHM CM

EQUI-RESISTIVITY CONTOUR MAP OBTAINED WITH A 20ft ELECTRODE SPACING

DAVIS-BESSE NUCLEAR POWER STATION

OAK HARBOR, OHIO

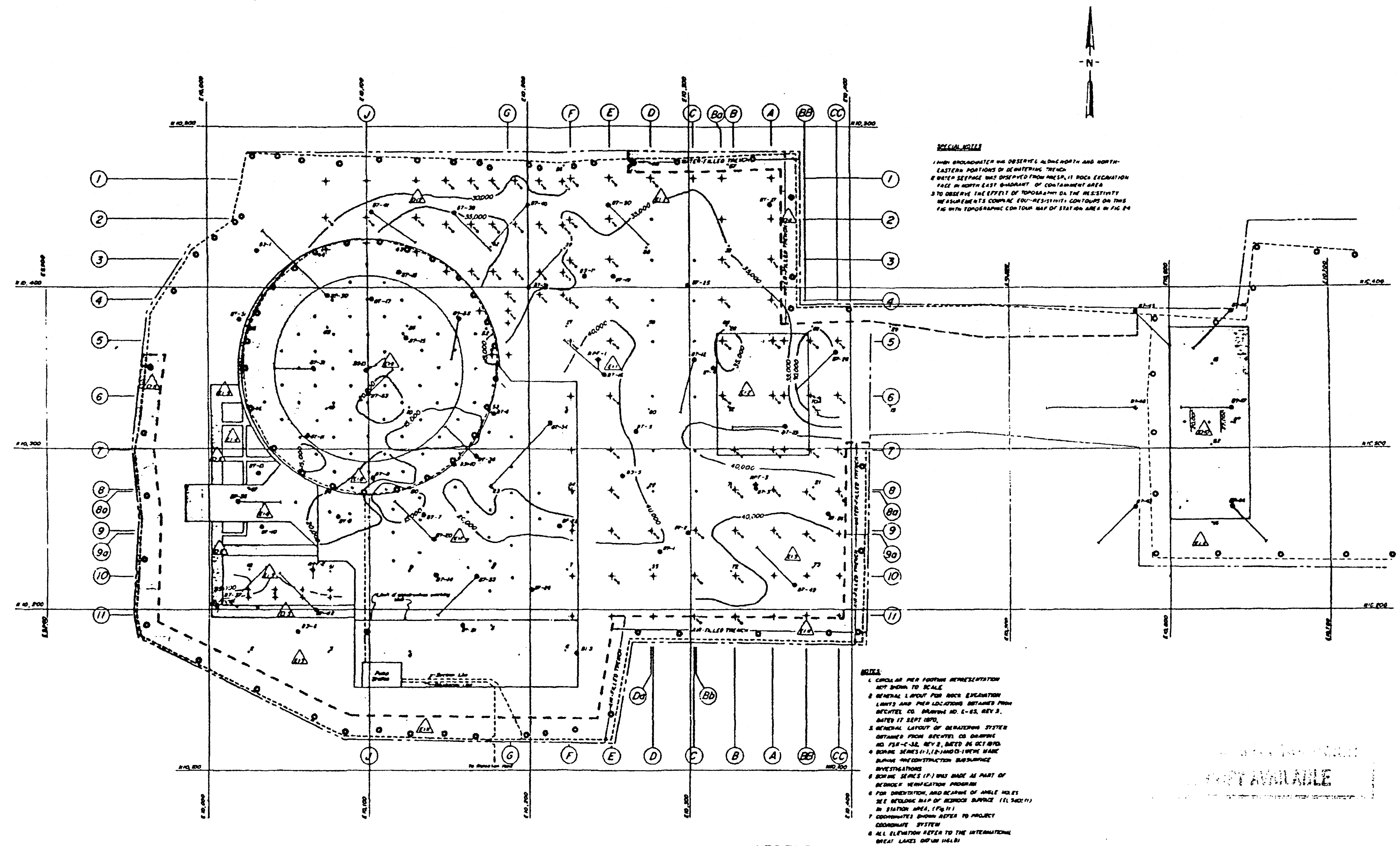
GRAPHIC SCALE, ft

WMAI 68-192

FIGURE 2C.5-37

REVISION 0

JULY 1982



LEGEND

- LOCATION OF TOE OF SOIL SLOPE
- LIMITS OF ROCK EXCAVATION IN BUILDING AREA
- LOCATION OF VERTICAL BORING 7-31
- LOCATION, BEARING AND HORIZONTAL PROJECTION OF ANGLE BORING 7-38
- LOCATION OF PRECONSTRUCTION ROCK PROBE 18
- LOCATION OF FOUNDATION GRADE ROCK PROBE 1
- LOCATION OF DEWATERING HEADER
- APPROX LIMITS OF DEWATERING TRENCH EXCAVATION
- DEWATERING SYSTEM WELL LOCATION
- (A) NORTH-SOUTH CONSTRUCTION CONTROL LINE
- (2) EAST-WEST CONSTRUCTION CONTROL LINE
- LOCATION OF PIER FOOTING
- ELEVATION SYMBOL
- LOCATION OF RESISTIVITY READING
- LOCATION OF RESISTIVITY READING IN PIER FOOTING
- 40,000 INFERRED EQUI-RESISTIVITY CONTOUR IN UNITS OF OHM-CM

ELEVATION NOTES

SYMBOL	AREA ELEVATION	REMARKS	SYMBOL	AREA ELEVATION	REMARKS
El 1	5580	EXIST ROCK SURFACE	El 10	5430	EXCAV ROCK SURFACE
El 2	5560	EXCAV ROCK SURFACE	El 11	5570	EXCAV ROCK SURFACE
El 3	5610	EXIST ROCK SURFACE	El 12	556 0	EXIST ROCK SURFACE
El 4	5560	EXIST ROCK SURFACE			
El 5	55625	EXCAV ROCK SURFACE			
El 6	54275	EXCAV ROCK SURFACE			
El 7	54175	EXCAV ROCK SURFACE			
El 8	53975	EXCAV ROCK SURFACE			
El 9	5260	EL OF EXCAV ROCK SURFACE AT CENTER OF BOWL SHAPE EXCAVATION			

EQUI-RESISTIVITY CONTOUR MAP OBTAINED WITH A 40ft ELECTRODE SPACING

DAVIS-BESSE NUCLEAR POWER STATION

OAK HARBOR, OHIO

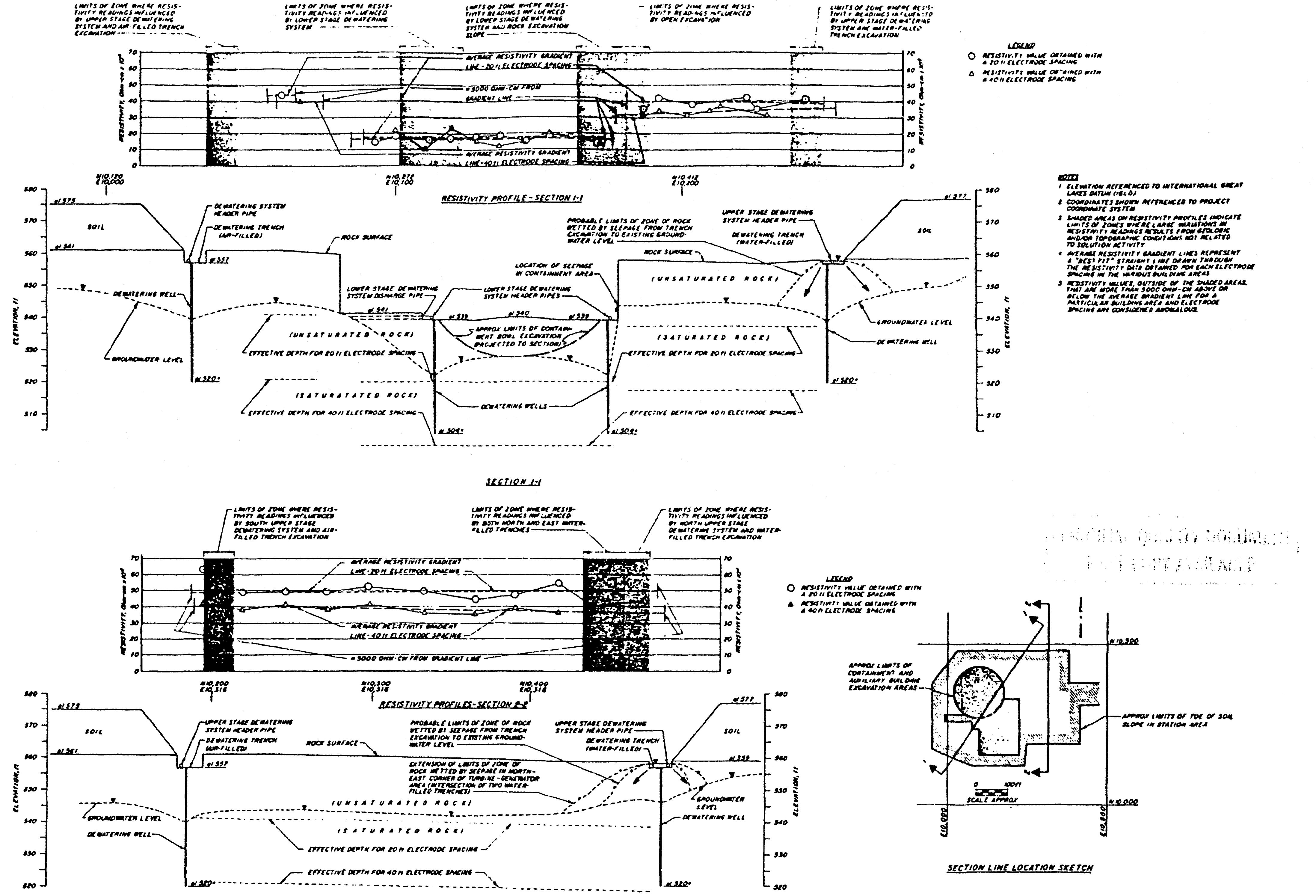
GRAPHIC SCALE ft

WMAI 68-192

FIGURE 2C.5-38

REVISION 0

JULY 1982



RESISTIVITY PROFILES ALONG TWO SECTION LINES IN THE STATION AREA

DAVIS-BESSE NUCLEAR POWER STATION

OAK HARBOR, OHIO

PROFILE SCALES

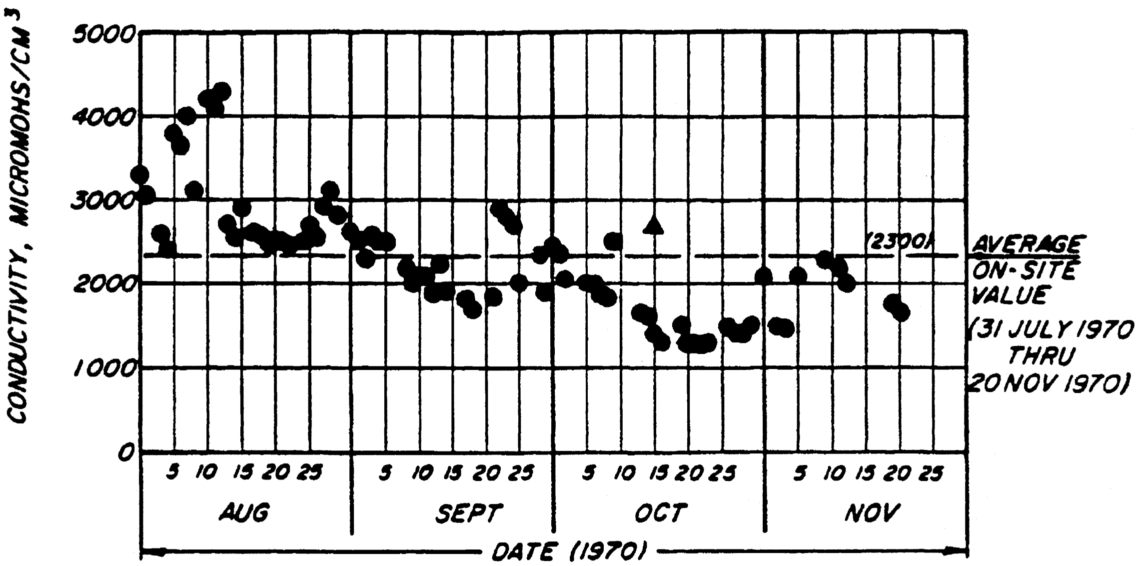
RESISTIVITY PLOT SCALES

WMAI 68-192

FIGURE 2C.5-39

REVISION 0

JULY 1982



LEGEND

● ON-SITE LABORATORY CONDUCTIVITY DETERMINATION

▲ OFF-SITE LABORATORY CONDUCTIVITY DETERMINATION

ARCHIVAL QUALITY DOCUMENT
BEST COPY AVAILABLE

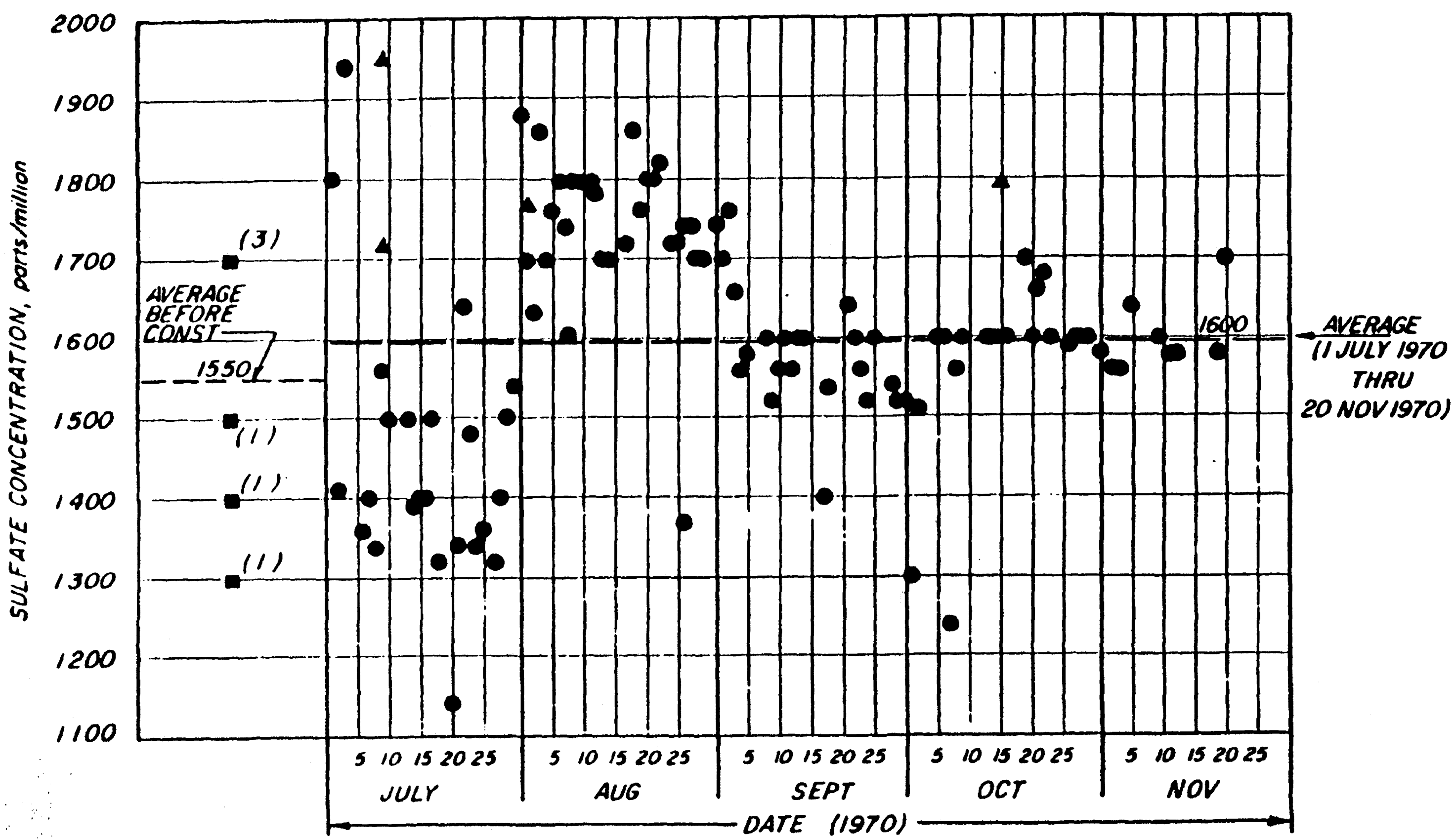
GROUNDWATER CONDUCTIVITY VERSUS TIME
(31 JULY 1970 TO 20 NOV 1970)

FIGURE 2C.5-40
REVISION 0
JULY 1982

WMAI 68-192

WMA 68-192

GROUNDWATER SULFATE CONCENTRATION VERSUS TIME
(1 JULY 1970 TO 20 NOV 1970)



LEGEND

● - ON-SITE LABORATORY SULFATE DETERMINATION-DURING CONSTRUCTION.
▲ - OFF-SITE LABORATORY SULFATE DETERMINATION-DURING CONSTRUCTION.
■ - OFF-SITE LABORATORY SULFATE DETERMINATION-BEFORE CONSTRUCTION
( ) - NUMBER IN PARENTHESES INDICATES THE NUMBER OF TESTS WITH SAME SULFATE CONCENTRATION.

FIGURE 2C.5-41
REVISION 0
JULY 1982

MEASURED DEWATERING SYSTEM DISCHARGE RATE VERSUS TIME
(6 JUNE 1970 TO 1 AUGUST 1972)

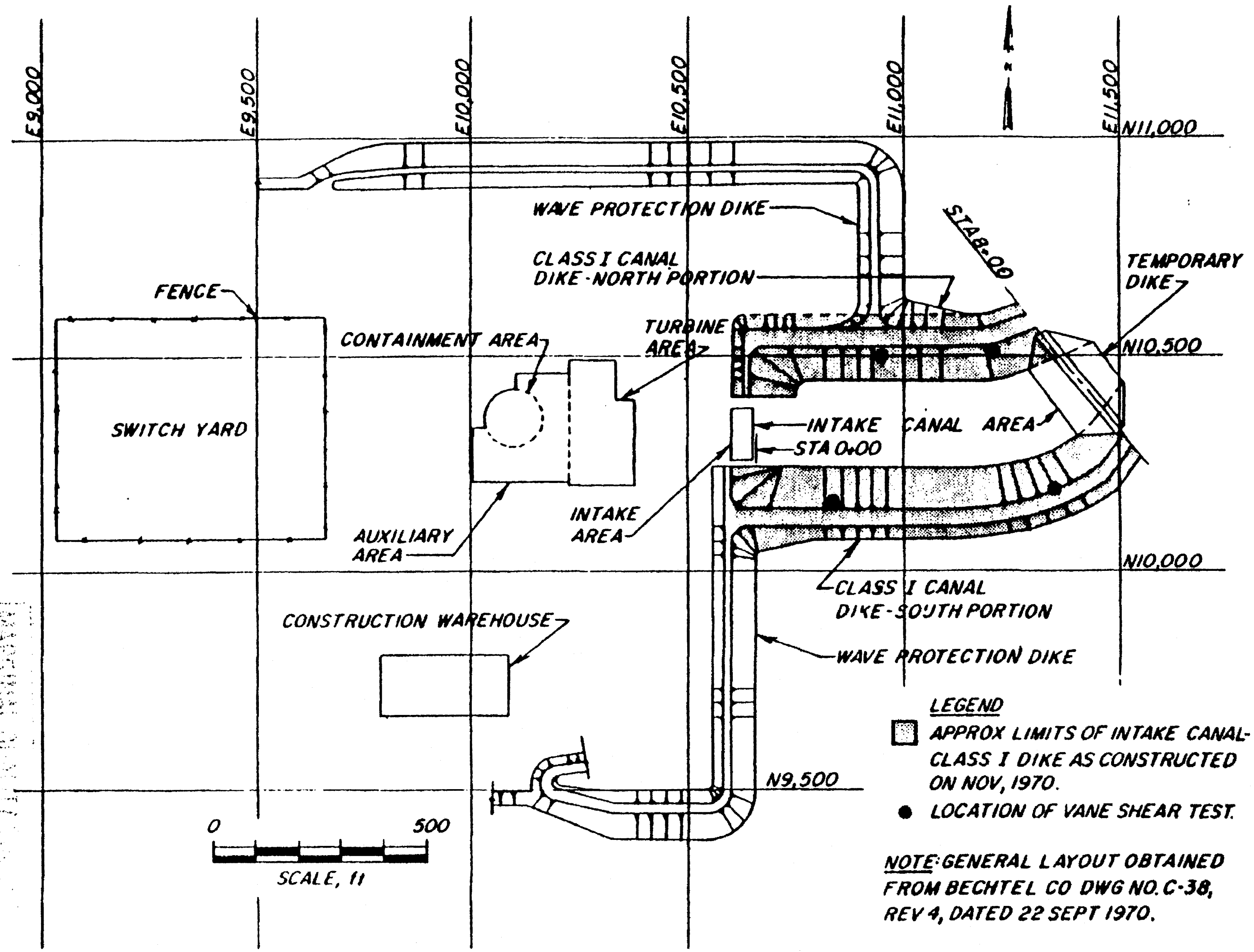
FIGURE 2C.5-42

REVISION 0
JULY 1982

WMAI 68-192

WMA168-192

INTAKE CANAL-CLASS I DIKE LOCATION PLAN

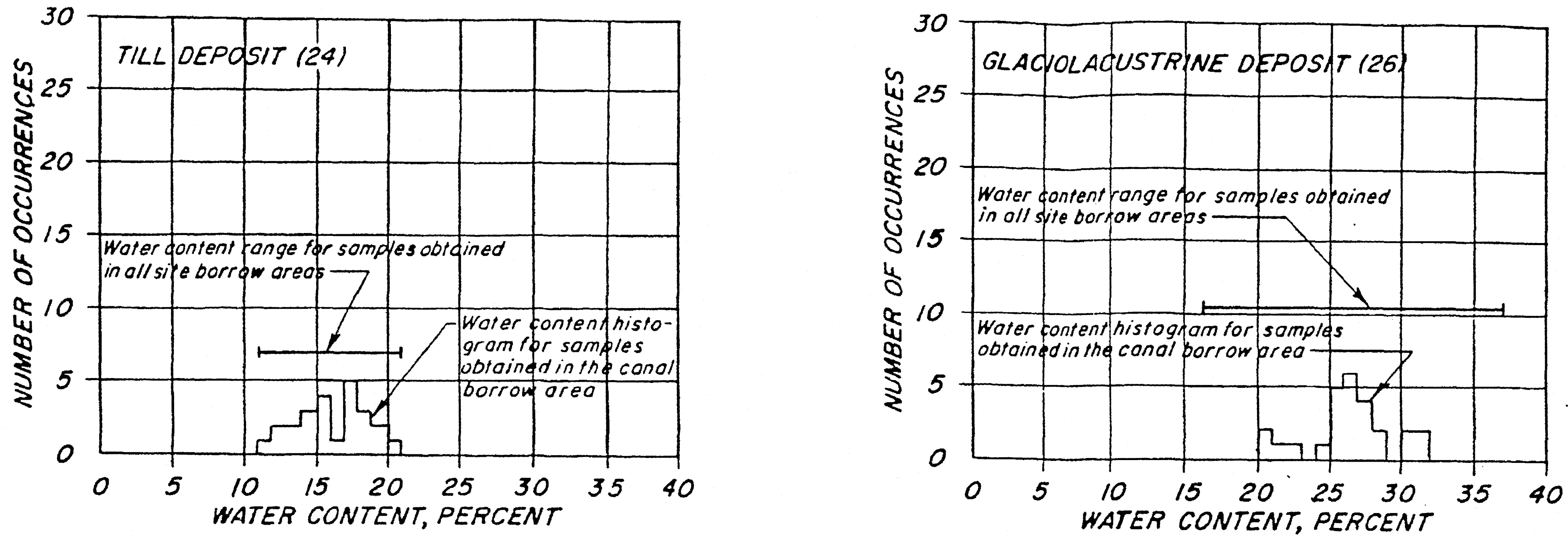


LEGEND

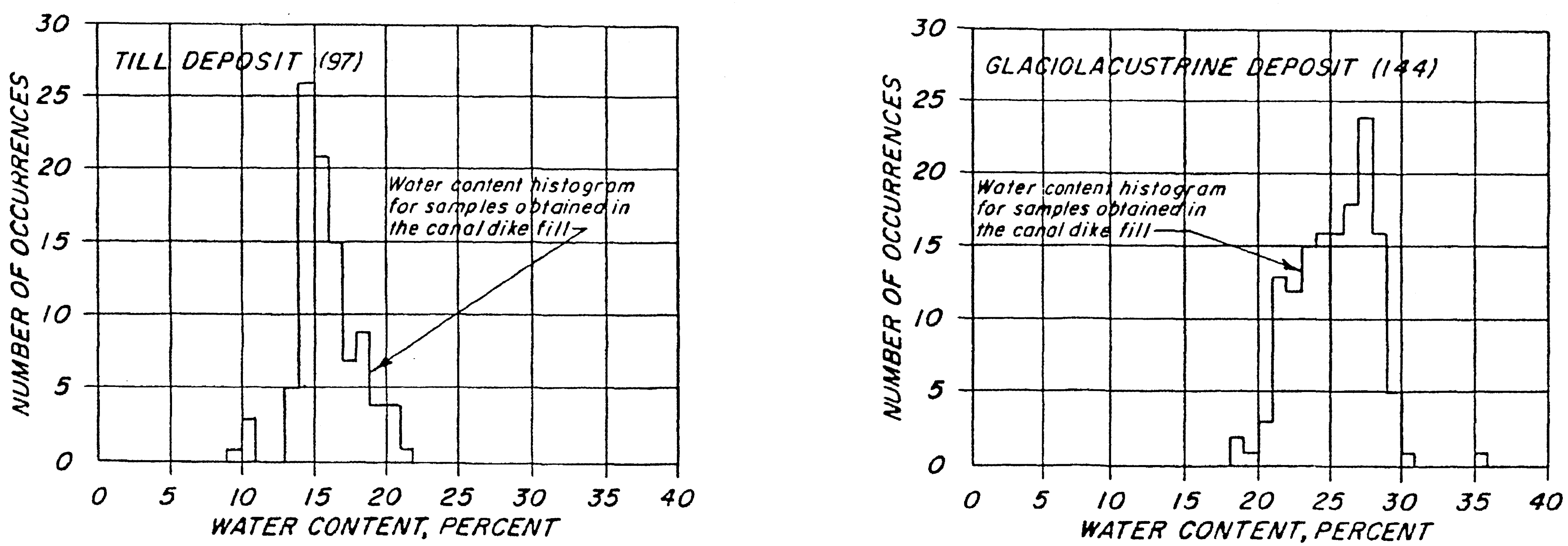
- APPROX LIMITS OF INTAKE CANAL-CLASS I DIKE AS CONSTRUCTED ON NOV, 1970.
- ● LOCATION OF VANE SHEAR TEST.

NOTE: GENERAL LAYOUT OBTAINED FROM BECHTEL CO DWG NO. C-38, REV 4, DATED 22 SEPT 1970.

FIGURE 2C.5-43
REVISION 0
JULY 1982



BORROW AREA WATER CONTENT RESULTS



INSITU CLASS I DIKE FILL WATER CONTENT RESULTS

NOTE:

NUMBER IN ( ) INDICATES THE NUMBER OF WATER CONTENT SAMPLES TAKEN IN THE INTAKE BORROW AREA OR FROM THE CLASS I DIKE FILL FOR EACH DIKE.

WATER CONTENT TEST RESULT HISTOGRAM - INTAKE CANAL CLASS I DIKES (26 AUGUST 1970 THROUGH 6 OCTOBER 1970)

WMAI 68-192

FIGURE 2C.5-44
REVISION 0
JULY 1982

CLASS I INTAKE FOREBAY DIKE LOCATION PLAN

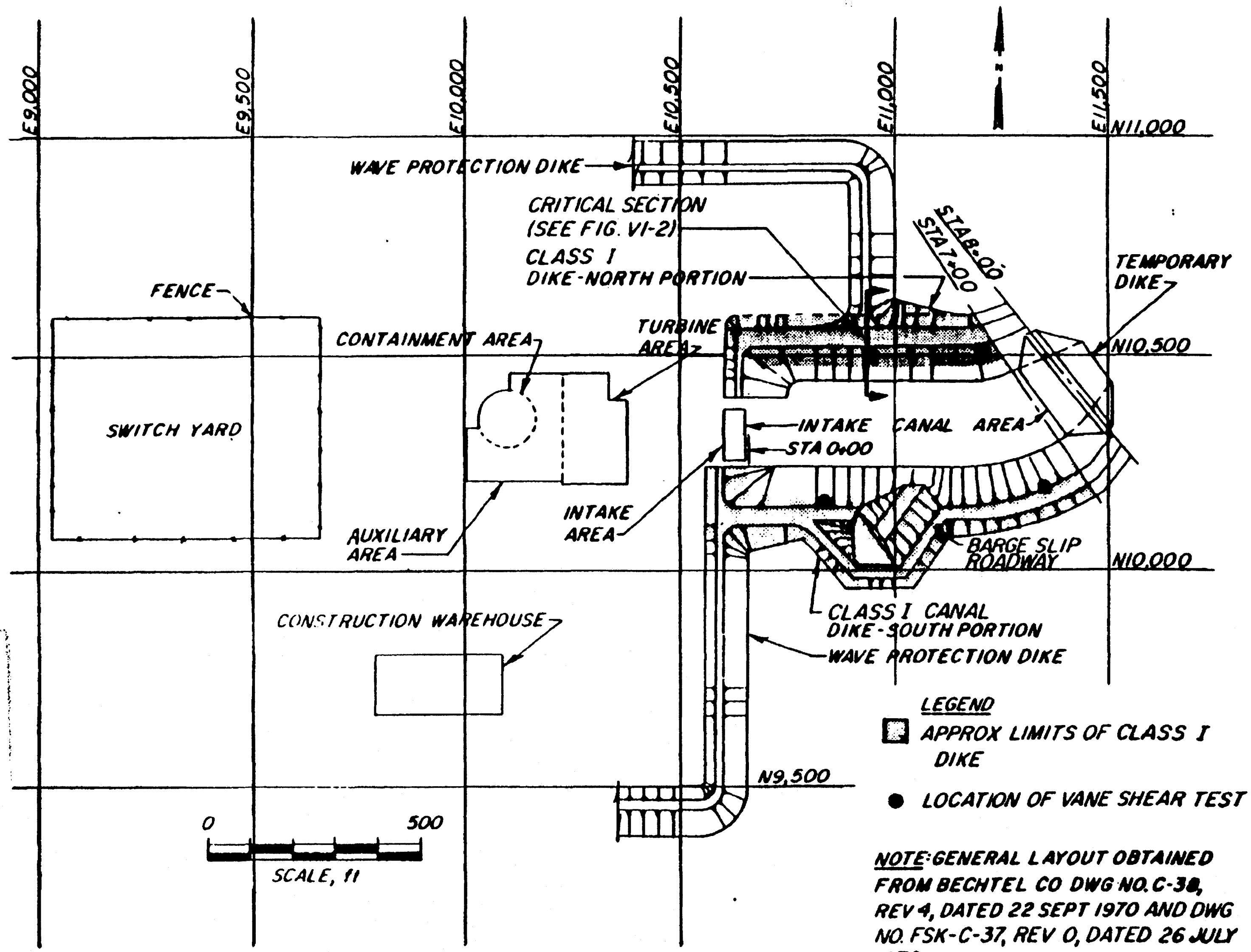
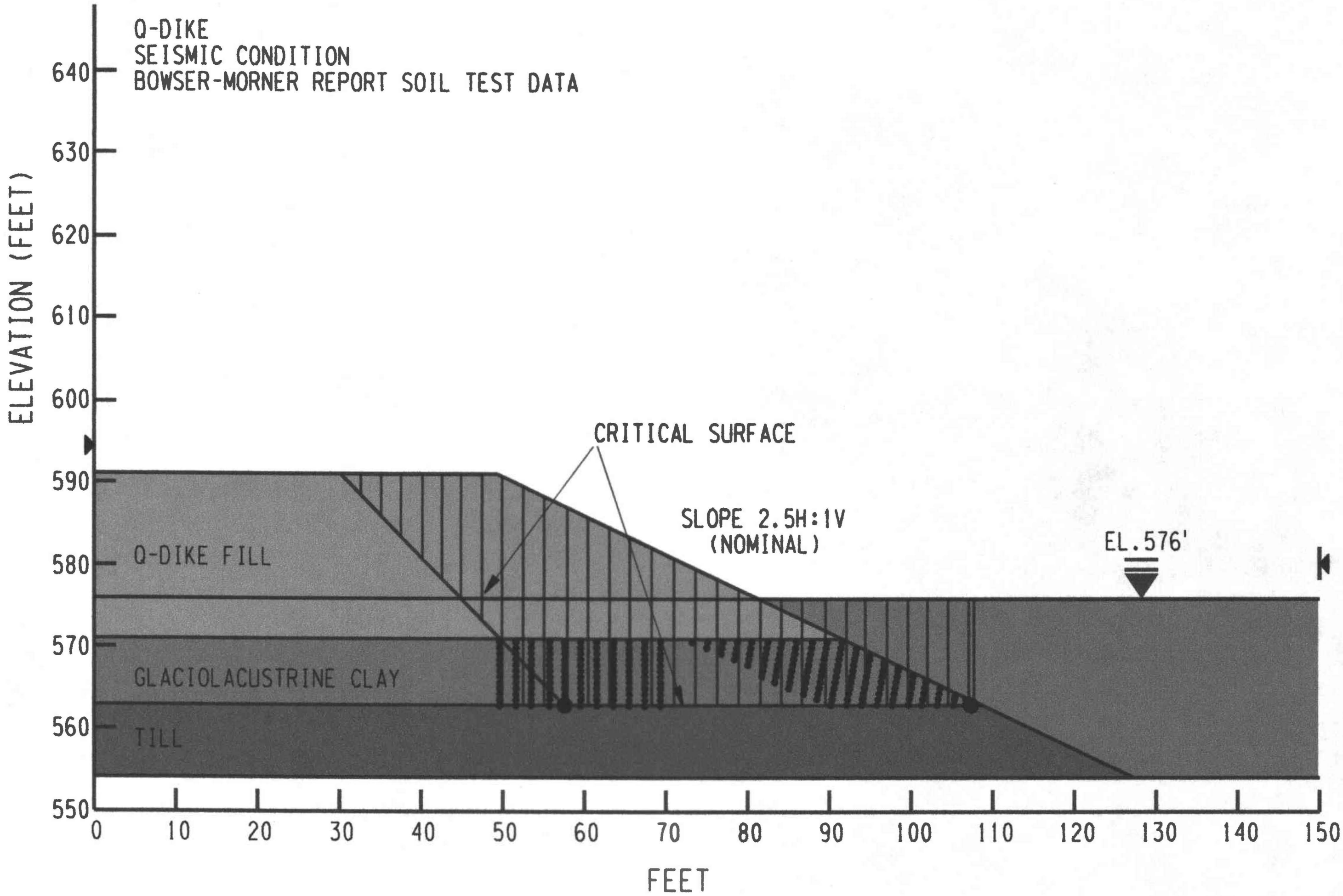


FIGURE 2C.6-1
REVISION 0
JULY 1982

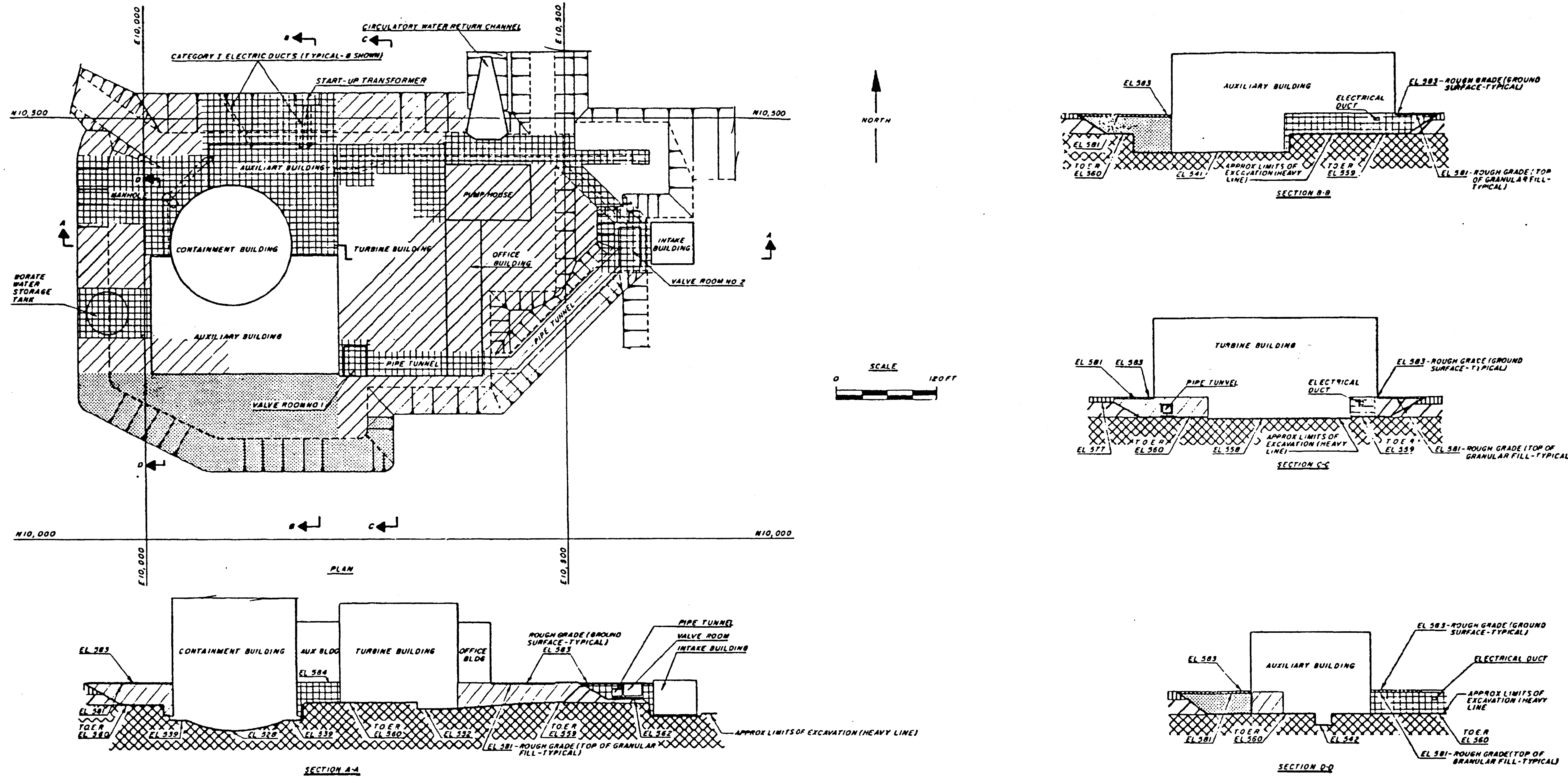


NOTE:
SOIL STABILITY ANALYSIS OF THE CLASS 1 INTAKE FOREBAY CANAL DIKES FOR THE EXTENDED DURATION (AS-BUILT OR CURRENT) CONDITION FOR MAXIMUM POSSIBLE EARTHQUAKE IS PERFORMED IN REFERENCE 59.

DAVIS-BESSE NUCLEAR POWER STATION
TYPICAL PROFILE OF CLASS 1 INTAKE
FORBAY CANAL DIKES
FIGURE 2C.6-2

REVISION 30
OCTOBER 2014

DB 02-18-14 DFN: G/USAR/UFIG2C62.DGN



NOTES

1. GENERAL LAYOUT OBTAINED FROM BECHTEL COMPANY DRAWING NO. C-65, REV NO. 1, NO DATE
2. ELEVATIONS SHOWN ARE REFERENCED TO INTERNATIONAL GREAT LAKES DATUM (IGLD).
3. COORDINATES SHOWN ARE REFERENCED TO PROJECT COORDINATE SYSTEM.
4. T.O.E.R. - TOP OF EXISTING ROCK SURFACE

LEGEND

- CLASS I GRANULAR STRUCTURAL FILL
- NON-CLASS I GRANULAR STRUCTURAL FILL
- NON-CLASS I GRANULAR GENERAL FILL
- NON-CLASS I EARTHEN GENERAL FILL
- IN SITU SOIL DEPOSIT
- IN SITU BEDROCK DEPOSIT

EXTENT OF CLASS I GRANULAR BACKFILL IN STATION AREA

WMAI 68-192

FIGURE 2C.6-3

REVISION 0
JULY 1982

APPENDIX 2D

VERIFICATION STUDY
PLATZMAN'S WIND SETUP MODEL FOR LAKE ERIE
WITH APPLICATION TO THE DAVIS-BESSE NUCLEAR PLANT

PREPARED BY
DAMES & MOORE
JOB NO. 7696-013-02

MARCH, 1975
REV. 12

FOR

THE TOLEDO EDISON COMPANY

APPENDIX 2D

REPORT VERIFICATION STUDY PLATZMAN'S WIND SETUP MODEL FOR LAKE ERIE WITH APPLICATION TO THE DAVIS-BESSE NUCLEAR PLANT FOR TOLEDO EDISON

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2D.1.0 Introduction

This report contains our verification study of Platzman's one-dimensional Wind Setup Model for Lake Erie with application to the probable maximum lake levels at the Davis-Besse plant site. A computer plot of Lake Erie geometry and representative field depths, showing the location of the lake gauging stations at Toledo and at Marblehead, is presented in Figure 2D.1-1.

All lake levels in this report are with respect to (WRT) the International Great Lakes Datum of 1955 (IGLD-1955) on which the mean level of Lake Erie is 568.6 feet. This datum is not to be confused with the New York Mean Tide Datum of 1935 (NYMT-1935) on which the low water datum of Lake Erie is 570.5 feet.

2D.1.1 Purpose

The purpose of this study is the verification of Platzman's one-dimensional Wind Setup Model for Lake Erie in order to determine if the previously calculated Probable Maximum Wind Setup resulting from the Probable Maximum Meteorological Event (PMME) is a conservative value. Platzman's model is a digital model which may be calibrated by comparing the computed value of wind setup to the measured value obtained from lake level recorders and adjusting the parameters of the model to yield conservative results. Platzman (ref. 1) performed a limited series of such tests and produced the calibration used in the original calculation of the Probable Maximum Wind Setup at the Davis-Besse site.

2D.1.2 Scope

The scope of this study includes the following:

a. Reduction of wind data obtained from several lakeside wind stations to a format compatible with Platzman model input for four selected storms.

b. Calculation of the wind setup value at Toledo by means of Platzman's model for each of the four storms.

c. Comparison of the calculated value with the measured value obtained from the lake gaging station at Toledo for each of the four storms.

d. Verification of the model.

e. Recalculation of the probable maximum wind setup at the Davis-Besse site using the verified model.

2D.1.3 Description of Platzman's Model

A complete description of Platzman's model is presented in Reference 1. Briefly, the input data are winds coded in the format of the Great Lakes Marine Forecasts. These are applied to a set of pre-computed response tables from which, by superposition, a hydrograph is obtained of consecutive three-hourly lake levels (relative to the prevailing mean lake level) at a particular location.

The response tables are constructed by numerical integration of a linearized hydro-dynamical Channel Model. The validity of such a model in the case of Lake Erie has been fully discussed by Platzman (ref. 1) and has been acknowledged for the Davis-Besse plant site.

The bases for the model are the equations of momentum and mass balance along the channel (or lake) axis characterized by Section area S(x), which are:

$$\frac{\partial Q}{\partial t} = -gS\frac{\partial \xi}{\partial x} + \frac{b}{\rho}(\tau_0 - \tau_1)$$

$$\frac{\partial \xi}{\partial t} = -\frac{1}{b}\frac{\partial Q}{\partial x}$$

where Q(x,t) is the volume transport through the section, ξ (x,t) is the mean water level across the section, ρ is the water density (assumed uniform) and τ_0 and τ_1, are respectively the mean surface and bottom stress across the section. The phrase "across the section" implies a mean taken transversely over the section width, b. The theoretical derivation of the equations and discussion of the approximations involved are given by Defant (ref. 2).

Of particular interest to this study is the method used to evaluate the surface and bottom stress. For the bottom friction the quadratic stress law, τ_1= kρ/u/u, is widely accepted where k (the value of which is found to be about 0.003 in work on tidal friction) is a friction coefficient and u is the velocity averaged over a section. However, the Platzman model is a linearized model in which the linear law τ_1= ρUu was adopted where U is a constant friction "velocity" which can be thought of as the product of the friction coefficient, k, and a scale value of u. As a result of tests, Platzman found that a value of 0.002 feet/sec gave best verification of the computed setup. (Based on a friction coefficient of 0.003, the scale value of u is 0.7 feet/sec.) As a consequence of the linear friction law, the value of bottom friction would tend to be underestimated and hence the calculated setup overestimated for situations in which the actual value of u exceeds the scale value of 0.7 feet/sec. This will in all probability be true for the case of the PMME.

For the surface or wind friction the expression τ_0 = $k\rho_a E$ is widely accepted where K is a friction coefficient, ρ_a is the air density, and E is the "effective wind-square." The "effective wind-square" for Lake Erie is E = W^2Cos (247.5°-d) where W is the wind speed and d the wind direction in degrees true North. The angle 247.5° represents the direction adopted for the channel axis of the lake.

The most important and sensitive parameter in the model is the proportionality factor K, since the wind shear is the primary driving force. The proportionality K should be a function of other parameters such as wind speed, the height of measurement of the winds, a characteristic length of surface roughness and stability and turbulence of the atmosphere. The form of the relationship which appears most frequently in the literature is $K = C_1 + C_2\left(1 - {}^{Wc}\!/_{W}\right)^2$ where C_1 and C_2 are experimental constants and Wc is a critical wind velocity. As the wind speed increases, the value of K approaches the sum of $C_1 + C_2$ asymptotically.

The critical wind velocity, Wc, varies from about 12 to 16 miles per hour. In reviewing the literature, Wilson (ref. 3) found that K = $3.79 \times 10^{-6} \pm 0.79 \times 10^{-6}$ for strong winds and K = $2.37 \times 10^{-6} \pm 1.26 \times 10^{-6}$ for light winds. Although comparison of various reported values of the proportionality K are quite scattered, there is a general tendency for K to be one constant as the wind speed is below a critical value and another constant as the wind speed approaches infinity. As most investigations have dealt with low wind speeds, extrapolation techniques have been used to obtain the necessary information at high wind speeds. Although experimental data in

the high wind region are sparse, there is more turbulence, wave breaking and energy dissipation in the high wind, hence, an ever increasing K is not possible.

Platzman adopted a constant value of K on the basis of limited tests. The value of 1.8×10^{-6} gave best results when used with the storm winds provided by the Great Lakes Marine Observations. The value of K was determined using the relatively low shoreline wind velocities available for calibration purposes. As a consequence, the constant proportionality factor and hence the value of the wind stress and the calculated setup would tend to be overestimated, especially for light winds.

Another simplification imposed on the Platzman model is that only a single wind is input for the lake as a whole at each time. The method used to determine the single input wind is not well defined by Platzman, but has been left to the discretion of the user. Hence the problem arises during the verification of the model to determine to what extent defects in the calculated setup can be attributed to the prediction model and to what extent to the input winds.

A shortcoming of the model is the fact that it cannot account for variations transverse to the axis of the lake. The possibility of cross-lake setup is real and often cannot be neglected. Platzman describes other difficulties that may be encountered in using the linearized, one-dimensional model. Nonetheless, the model may be calibrated to produce useful results as long as the limitations of the results are clearly understood.

2D.1.4 Verification of the Model

Method:

The method used to verify Platzman's model was to develop a relationship for the difference between the peak lake levels predicted by the model and the corresponding actual (measured) levels, as a function of wind speed. The model calibration coefficients determined by Platzman, were used in the calculation of lake levels. The differences between the calculated and measured peak lake levels during four recorded major storms were analyzed to define the desired relationship.

Storm Wind Data:

Initially, wind data for the verification of the Platzman model were taken from various airport weather stations surrounding Lake Erie for four different storms, April 26-28, 1966, November 13-15, 1972, March 16-18, 1973, and April 7-10, 1973. After vectorially summing and averaging the winds from the various stations to get a single wind at each time for the lake as a whole, the calculated setup at Toledo was found to be much less then the measured values. This difference was attributed to the low magnitude overland winds, reduced by local topography and buildings, which were not representative of conditions over the body of the lake.

Therefore, wind data from the lakefront coast guard stations of Buffalo, New York; Ashtabula, Cleveland, Marblehead and Toledo, Ohio were used in the verification calculations. Although the data were incomplete in that all stations did not report for each of the four storms mentioned above, the winds from the reporting stations were vectorially summed and averaged as in the previous case. The calculated setup at Toledo was lower than the measured setup; however, there was better agreement than was obtained in the initial attempt. The differences between the computed and the measured setup values were attributed to the unequal distribution of the stations around the lake. The locations of the stations at Buffalo and to some extent at Toledo

are such that wind speeds are reduced by the effect of land. Apparently the various stations should not have been given equal weight in the averaging procedure.

Good agreement between the computed and the measured setup values was obtained when the wind data from Ashtabula Coast Guard Station was used. This station represented the best single average resultant wind because it is located along the shoreline midway between Toledo and Buffalo, and it is less protected, thereby minimizing land effects on the wind. In addition, the data at Ashtabula is the only station which is complete for all four storms to be considered in the verification. Platzman (ref. 1) indicated that wind data from Clear Creek, Ontario, showed good correlation with observed setup. Unfortunately, the Clear Creek weather station was removed from the meteorological network some time ago. Ashtabula, however, is located nearly opposite Clear Creek on Lake Erie and its exposure to overwater winds should be similar. Therefore wind data from Ashtabula Coast Guard Station was used for verification of the model.

Wind data is recorded in the Ashtabula Marine Coastal Weather Log at intervals of approximately 2 hours. These data were plotted and graphically averaged over 3-hour intervals as a first stage in the reduction of wind data for input to the Platzman model. The wind data discussed in the following paragraphs are these 3-hour intervals' average values.

The first storm used in the verification occurred during April 26-28, 1966 with no data available for the Buffalo Station. The Ashtabula winds peaked on the 26th between 1300 and 1900 (EST) with a speed of 40 knots from the northeast (see Figure 2D.1-2). At Toledo, very high winds to 45 knots from the northeast were present between 0200 and 1800 (EST) on the 27th (see Figure 2D.1-3). A second but lesser peak appeared in the wind at Ashtabula on the 28th near 1300 (EST) with a speed of 26 knots. By that time, the wind had veered from the northeast to the southeast.

The second storm occurred during November 13-15, 1972 and was characterized by very high winds at Ashtabula which peaked at 50 knots from east-northeast near 0700 (EST) on the 14th (see Figure 2D.1-4). Cleveland, Marblehead and Buffalo generally peaked at about the same time period with much lower wind speeds also from the northeast. The winds changed direction between 0700 and 1200 on the 15th and began to blow from the northwest, with a maximum speed of 17 knots at Ashtabula. No data were available for the Toledo Station.

The third storm of March 16-18, 1973 was represented by data from Ashtabula and Buffalo. Ashtabula had the greater winds which peaked first at 2200 (EST) on the 16th with speeds slightly over 30 knots from the northeast. The winds changed direction between 0800 and 1600 on the 17th and began to blow from the northwest. The Ashtabula winds peaked for a second time at 1800 (EST) on the 17th with speeds of 45 knots from the west (see Figure 2D.1-5).

Data for the final storm, which occurred during April 7-10, 1973, was available for Ashtabula and Buffalo. The Ashtabula winds reached a maximum speed of 30 knots from the northeast at about 0700 (EST) on the 8th (see Figure 2D.1-6). At 1800 (EST) on the 9th they began to veer from the northeast to a westerly direction. Secondary peaks appeared in the winds at about 1900 (EST) on the 9th and at about 2000 on the 10th at which time the wind had increased to 29 knots from the west-northwest. Supplementary wind data from the Detroit River Light Station indicated that the highest wind speeds in the vicinity of Toledo reached 34 knots from the northeast, occurring at 1600 (EST) on the 9th (see Figure 2D.1-7).

Three-hour average wind speeds and directions at Ashtabula were interpolated as shown on Figure 2D.1-2, Figure 2D.1-4, Figure 2D.1-5 and Figure 2D.1-6 and reduced to their respective codes for input to Platzman's model.

Lake Level Data:

The physical location of a lake level gauge is very important, for a location subject to local disturbances such as harbor oscillations, reflections from breakwaters, etc., can cause significant errors in the gauge records. Some of the gauges on Lake Erie are well situated, while others are not properly located to give the general, lake level fluctuation. The geographical location of the Toledo Gauge and the Marblehead Gauge are shown on Plate 1. Both gauging stations are located on the southern side of the western end of the lake with Toledo excellently situated at the extreme western end to record maximum setup levels. These gauges record the general lake level and are not adversely affected by local disturbances.

The Toledo Gauge and the Marblehead Gauge belong to the U.S. Lake Survey network of recording stations. The Toledo Gauge is located on the U.S. Army Corps of Engineers' property at the mouth of the Maumee River. The gauge well is connected to a pipeline which extends approximately 300 feet from the shoreline into Maumee Bay. The Marblehead Gage is situated in a Coast Guard Slip located on the northern side of the Marblehead Peninsula.

The hourly measured lake levels at Toledo and at Marblehead for each of the storms used in the verification study are presented in Figures 2D.1-8 through 2D.1-11. The peak water level fluctuations at Marblehead were consistently less extreme than the fluctuations at Toledo. This is due to the relative location of the two gauges, with the Toledo Gauge situated so as to record the longitudinal setup at the extreme end of the lake.

For the purpose of verification of the Platzman model, the measured lake levels at Toledo were used. This was consistent with Platzman's original intention (and calibration) of predicting the lake levels at Toledo.

2D.1.5 Results

April 26-28, 1966:

A significant difference existed between the calculated setup and the measured setup at Toledo for this storm. This discrepancy apparently was due to the relatively strong crosswinds recorded at Toledo on the 27th which resulted in significant cross-lake setup at Toledo. Due to the geometry of the lake, the channel axis of the western basin is directed 290 degrees true north which is deflected northward of the axis of the main basin. Hence it is possible for winds from the northeast to act as crosswinds over the western basin. This situation is aggravated by the geometry of Maumee Bay and the location of the Toledo Gauge at the mouth of the Maumee River. The coincidence of the peak measured setup at Toledo with the peak wind speeds at Toledo, while the wind speeds at Ashtabula had already begun to drop, affirms the conclusion that the poor agreement was due to the crosswind condition. The one-dimensional Platzman model could not account for this occurrence, as it calculates the longitudinal component of setup only. Therefore, the cross-lake component of setup was estimated using methods outlined in Reference 4 and the Toledo winds, and subtracted from the measured setup at Toledo. Once this was done the two hydrographs agreed more closely, with the calculated peak setup being conservative (see Figure 2D.1-8).

November 13-15, 1972

Good agreement was achieved between the measured setup and the calculated setup for this storm. However, due to an exceptionally high peak in the wind speed which was recorded as 50-60 knots at 0615 (EST) on the 14th in the weather log, the peak 3-hour average wind of 46 knots shown on Figure 2D.1-4 may be somewhat unrepresentative. Although the peak wind speed was of short duration (the record at 0415 (EST) shows 31 knots and at 0817 (EST) shows 35-45 knots), a minor shift in the averaging time scale would result in a slightly higher peak 3-hour average wind speed of 48 knots. The slight change in peak wind speed would be of little consequence except for the fact that Platzman's model is a discretized model, that is, one in which input parameters are lumped into groups, and response calculated for the mean value of each group. The peak wind speed of 46 knots falls into Category 6, while the peak speed of 48 knots falls into Category 7. The hydrographs for both cases are shown on Figure 2D.1-9. The difference in peak setup calculated due to the change in peak wind speed from category 6 to 7 is 2.0 feet, with the difference decreasing rapidly on either side of the peak. The essential result is that the best estimate of the calculated peak setup is the average of the two cases, as the best estimate of the peak 3-hour average wind speed falls on the borderline between the respective wind categories. The prominent peak in the calculated hydrograph was conservative and occurs at nearly the same tine as the measured peak (see Figure 2D.1-9).

March 16-18, 1973:

The shape of the measured hydrograph was closely approximated by the calculated results for this storm. Once again the calculated peak setup was conservative (see Figure 2D.1-10).

April 7-10, 1973:

The first prominent peak of the calculated and measured hydrographs agreed fairly well, with the calculated setup being conservative. There was a second prominent peak, however, in which the measured value was greater than the calculated value. This deviation was due to a crosswind condition similar to that which existed during the April 1966 storm. The peak of the windspeeds at the Detroit River light station coincides with the second peak in the measured setup at Toledo, and incidentally with a second peak in the wind speed at Ashtabula. Although the winds at Ashtabula were beginning to veer from the main axis of the lake at the time of the second peak, they still produced a significant contribution to the longitudinal component of setup, as is apparent in the calculated hydrograph. The cross-lake component of setup was estimated as in the case of the previous storm, and subtracted from the measured setup at Toledo. Once this was done, the difference between the secondary peaks was reduced considerably; however, it could not be eliminated entirely (see Figure 2D.1-11).

In the above comparisons, the calculated setup which was relative to the prevailing lake level was algebraically added to the mean monthly lake level as reported in Reference 5. The results of the comparison of peak wind setup and setup elevations, using Platzman's Model and the lake level gauge at Toledo, Ohio, are summarized in Table 2D.1-1.

2D.1.6 Discussion

The comparison between the calculated and the measured hydrographs at Toledo shows that the Platzman model, using the wind data from Ashtabula and the calibration constants provided by Platzman, can adequately predict the longitudinal component of the setup hydrograph.

The times of the primary peak and the subsequent peaks are in agreement. The magnitude of the primary calculated peak is conservative in all storms. The magnitude of the second prominent peak is approximate when allowance is made for the crosslake component of setup. Comparison of the magnitude of the subsequent peaks, which are a consequence of seiching resulting from the first or second peak, is of little significance. Complete agreement between the measured and calculated hydrographs is not possible, because the simplifying assumptions inherent in Platzman's one-dimensional model cannot account for the full complexity of the lake setup problem, including two-dimensional setup and seiching. This is especially true following the initial setup peak, for the storms used in the verification.

The oscillating characteristics of Lake Erie have been studied, both theoretically (refs. 8 and 9) and with prototype data (ref. 7). Lake Erie is renowned for the magnitude of its seiches (inertial oscillations of a water body which persist after the external forces have ceased to act). The amplitude of the seiche diminishes rapidly with each subsequent oscillation. The winds often continue to blow after the maximum setup has been reached at Toledo, but with much reduced force, so that there continues to be some component of wind setup. Although a small wind setup persists, the force of the wind is reduced sufficiently to permit the occurrence of inertial oscillations. In addition to inertial oscillation, the lake is also susceptible to forced oscillation which can result in resonant amplification of the water surface fluctuations. In situations where the driving force varies in a periodic manner, and the period of the force is close to a natural period of oscillation of the lake, the amplitude of successive oscillations can increase, rather than decrease.

The calculated and measured hydrographs demonstrate the complexity of the oscillating characteristics of Lake Erie. Therefore, the actual verification of Platzman's model is based on comparison of the first well defined peak in setup occurring after a period of relatively undisturbed lake levels. The secondary peaks, which in certain cases were influenced by cross wind conditions, were also affected by lake oscillations. During the storm of April 7-10, 1973, the wind speeds at Ashtabula, Ohio, exhibited three distinct peaks occurring at intervals of 28 to 30 hours. This interval coincides with (twice) the fundamental natural period of oscillation of Lake Erie, which is approximately 15 hours (ref. 9). A portion of the difference between the measured and calculated secondary peaks not attributable to crosswind effects, results from the lake oscillations. Although Platzman's model is able to simulate the fundamental mode of oscillation, there exist other modes of oscillation with shorter periods which can be significant and which are not predicted by the model. The modes of oscillation of Lake Erie and their respective periods are discussed in detail in Reference 9.

In order to demonstrate certain features of the oscillating characteristics of the lake, the hourly lake levels recorded at Toledo and Cleveland, Ohio, and Buffalo, New York, have been plotted for the storm of April 7-10, 1973 on Figure 2D.1-12. The time interval between successive prominent peaks is approximately 15 hours, and the Toledo and Buffalo records are nearly 180 degrees out of phase, clearly indicating the presence of the fundamental mode of oscillation. Cleveland, being close to the node, exhibits relatively low amplitude oscillation in phase with the Toledo record, as expected. The fundamental mode shape derived in Reference 9 defines the ratio of the difference in peak water level between Toledo and Cleveland to the difference between Cleveland and Buffalo.

The corresponding ratio at the times of maximum and minimum water levels can be obtained from Figure 2D.1-12 for comparison, in order to determine if the lake's surface was consistent with the fundamental mode shape. If not, the lake level must have been affected by disturbances (i.e. crosswind conditions, local oscillations, etc.) which could not have been predicted by the Platzman model. The results, summarized in Table 2D.1-2, show that at the

time of the second peak, 1600 (EST) on the 9th the lake level at Toledo was abnormally high, considering only the fundamental mode. The possibility that crosswind conditions and higher modes of oscillation are responsible for the abnormally high lake level is substantiated by the wind data and the distinct higher frequency fluctuations which appear in the hydrographs at Toledo, Cleveland and Buffalo following the second peak.

Two parameters were used to quantify the difference between the measured and the calculated peak setup elevations. First, the algebraic difference (in feet) between the maximum values of the primary peak setup elevation on the measured and calculated hydrographs were plotted versus the maximum three-hour average wind speed (see Figure 2D.1-13). Second, the net area (in feet-hours) between the measured and calculated hydrographs for a six-hour band encompassing both peaks was plotted versus the maximum six-hour average wind speed (see Figure 2D.1-13). Both techniques show a similar trend. The conservatism of the calculated peak setup elevation increases with increasing maximum wind speed. This statement is valid only for longitudinal setup and is subject to all other restrictions inherent in the Platzman model as discussed previously.

2D.1.7 Recalculation of the PMME Wind Setup at Toledo PMME Winds

The wind data used to calculate the setup at Toledo produced by the PMME were obtained from the table of probable maximum wind estimates (over water wind speeds) supplied by the AEC[1]. The spatial variation in wind speed over the lake was not considered in this calculation but only the temporal variation to simulate the conditions at Ashtabula under which the Platzman model was calibrated. So the values would more closely simulate the shoreline winds as used in the verification of the model, the over water PMME wind speeds were reduced by a factor of 0.89 (ref. 6). The resulting peak six-hour duration PMME wind speed is 77 knots. This value compares favorably with the peak speed of 77.6 knots used in the previous calculation of setup when the over water PMME winds were averaged along the lake axis. The PMME wind speeds were reduced to code for input to the Platzman model (see Figure 2D.1-14).

The wind direction was assumed to coincide with the axis of the lake to produce the maximum setup at Toledo.

2D.1.8 PMME Wind Setup at Toledo

During one time interval the PMME wind speeds exceeded Platzman's Category 8 winds, but no Category 9 response values were given in Platzman's tables. Therefore, the given Category 5 response values were extrapolated by multiplying by 4. This is an approximation based on the fact that the average of the extrapolated Category 9 winds is double the average of Category 5 and that doubling the wind speed will quadruple the wind stress and the calculated setup. This is a property of the Platzman linearized model.

The recalculated PMME setup hydrograph at Toledo is shown on Figure 2D.1-15. Also shown on Figure 2D.1-15 is the original Dames & Moore PMME hydrograph. The discrepancy between them may be ascribed to two factors; first, the difference in interpretation of the PMME wind field, and second, the difference in extrapolation of Category 9 wind response values.

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[1] Meeting with Dwight Nunn, then of AEC, July 17, 1970, participants were Dames & Moore, Toledo Edison, Detroit Edison, Bechtel and AEC.

The original extrapolation of the Category 9 response was made to 68 knots even though the maximum single PMME wind over the whole lake in the original analyses was 87 knots (100 miles per hour). In the present case the extrapolation was made to 74 knots while the maximum PMME wind was 77 knots. The present extrapolation is considered more conservative.

On Figure 2D.1-15 is shown a hybrid hydrograph calculated with the present PMME wind input and using the original Category 9 extrapolation. The resulting peak setup is essentially equal to the original peak setup. This shows that the discrepancy between the original peak setup and the recalculated peak setup is nearly entirely due to the difference in the extrapolated Category 9 wind response.

The recalculated maximum PMME wing setup at Toledo is 13.4 feet. Referring to Figure 2D.1-13 for a maximum three hour average wind speed of 74 knots, the model is estimated to compute a peak setup at Toledo two feet above the value which would be measured. Therefore, the maximum PMME wind setup with respect to wind blowing along the longitudinal axis of the lake actually should be 11.4 feet at Toledo, the 2.0 foot difference being the verified over-prediction of the model.

2D.1.9 PMME Wind Setup at the Davis-Besse Plant Site Reduction from Calculated Toledo Setup

The Davis-Besse plant site is located approximately 20 miles east of Toledo on the southern shore of Lake Erie. The plant site is between the point of maximum setup on the lake axis (Toledo) and the point of zero setup or nodal point (near Ashtabula). Hence, it seems reasonable to allow a reduction in maximum setup at Toledo to account for the relative position of the plant site along the lake axis. An adjustment factor of 0.8 was used to correct the wind setup at the Toledo gage to the Davis-Besse site. This adjustment factor was previously specified by the AEC[1].

The average peak setup height at Marblehead is 0.70 of the peak setup height at Toledo when the latter is corrected for cross-lake setup (see Table 2D.1-3). An unusually high ratio of 0.87 for the March 1973 storm resulted from cross-lake winds which setup the lake level at Marblehead relative to Toledo. The wind records for Gibraltar, Michigan, at the western end of the lake, show that during the time of peak setup the winds had veered from the northeast to the northwest resulting in cross-lake setup at Marblehead. For this reason the 0.87 value is not used in the average peak setup height ratio for Marblehead.

The average peak setup height ratio calculated for Marblehead exceeds the peak setup height ratio predicted for Marblehead by theoretical calculations based on a 2-dimensional study of seiching in Lake Erie (ref. 9). As the Marblehead peak setup heights were not corrected for cross-lake setup, the calculated ratio is conservative. In addition, the calculated ratio may be biased upwards by possible increased water levels at Marblehead as the islands adjacent to the Marblehead Peninsula undoubtedly constrict flow during longitudinal setup.

The adjustment factor of 0.8, specified by the AEC, for the Davis-Besse site is realistic in light of the conservatism in the calculated ratio for Marblehead. Theoretical calculations based on the model study (ref. 9) indicate the adjustment factor of 0.8 (see Figure 2D.1-16) is conservatively high for the Davis-Besse site. Accurate records of historical high water levels at the site, which may be used to verify the adjustment factor, do not exist.

The adjustment factor of 0.8 results in a maximum longitudinal wind setup at Davis-Besse of 9.1 feet.

2D.1.10 Allowance for Transverse Seiche

Due to the possibility of cross-lake setup occurring simultaneously with longitudinal setup at the Davis-Besse, the maximum longitudinal PMME wind setup of 9.1 feet at the Davis-Besse site is increased by 1.0 foot.

The value of 1.0 foot for cross-lake setup, also termed "transverse seiche" was derived from a report by I.A. Hunt of the U.S. Lake Survey Center, entitled, "Wind, Wave Setups and Seiches on Lake Erie." Hunt reported that a "transverse seiche" of 0.8 feet has been recorded in the western basin of Lake Erie. The AEC[1] has previously specified that a 1-foot transverse seiche be used at the Davis-Besse site.

The resulting maximum PMME setup, at the Davis-Besse site is 10.1 feet, which includes the allowance for transverse seiche.

2D.1.11 Conclusions

Verification of the Platzman One-Dimensional Wind Setup Model for Lake Erie:

The Platzman one-dimensional wind setup has been verified using four storms producing peak setup at Toledo. The model 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. The verification is valid for input winds measured at the Ashtabula Coast Guard Station. The model is valid for setup along the lake's longitudinal axis only. The verification for two of the storms indicated that cross-lake wind setup can be significant, and should be considered.

The verification for one storm indicated that seiching effects can be significant for secondary peaks and should also be considered.

The conservatism of the model in predicting the longitudinal setup increases with increasing wind speed. For a maximum three hour average wind speed of 74 knots the model is estimated to compute a longitudinal wind setup at Toledo two feet above the value which would be measured. However, an allowance should be made for the possibility of cross-lake setup occurring simultaneously with longitudinal setup at Toledo.

In addition, an allowance should be made for the possibility of higher order seiching occurring coincidentally with a secondary longitudinal setup peak. As seiching must occur following the initial setup peak, it need not be included in the primary peak PMME wind setup at the Davis-Besse site. Its inclusion in the second peak is of negligible consequence because the second peak PMME lake level is nearly 8 feet lower than the primary peak.

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[1] Meeting with Dwight Nunn, then of AEC, July 17, 1970, participants were Dames & Moore, Toledo Edison, Detroit Edison, Bechtel and AEC.

Maximum PMME Wind Setup at the Davis-Besse Site:

The maximum PMME wind setup at the Davis-Besse site based on the results of the verification of Platzman's model is 10.1 feet. This value is based on a verified maximum longitudinal PMME wind setup of 11.4 feet at Toledo, a 20 percent reduction in the Toledo setup to allow for the relative position of the Davis-Besse site along the lake axis, and a 1.0 foot increase to allow for the possibility of transverse seiche occurring simultaneously with the longitudinal setup.

The maximum probable stillwater elevation for the Davis-Besse site is 583.5 feet IGLD-1955. This level is based on a mean lake level of 568.6 feet IGLD-1955, maximum monthly lake level of 573.4 feet IGLD-1955 and the maximum PMME wind setup of 10.1 feet.

The maximum probable water elevation given in the PSAR is 583.7 feet IGLD-1955.

[1] Meeting with Dwight Nunn, then of AEC, July 17, 1970, participants were Dames & Moore, Toledo Edison, Detroit Edison, Bechtel and AEC.

TABLE 2D.1-1

Summary of Calculated Peak Wind Setup and Setup Elevations Using Platzman's Model and Measure Peak Setup at Toledo, Ohio for Four Storms

	PEAK SETUP ELEVATIONS (WRT IGLD-1955 Datum)		PEAK SETUP (WRT Mean Monthly Lake Level)	
STORM DATE	CALCULATED	MEASURED	CALCULATED	MEASURED
	574.0	573.1	3.7	2.8
April 26-28, 1966	573.6[1]	573.3[2]	3.3[1]	3.0[2]
Nov. 13-15, 1972	577.3	576.0	5.2	3.9
March 16-18, 1973	575.5	574.8	2.6	1.9
	575.8	574.9	2.4	1.5
April 7-10, 1973	574.7[1]	575.5[2]	1.3[1]	2.1[2]

Note: Setup refers to longitudinal component only
[1] Second Peak
[2] Second peak corrected for cross-lake component

TABLE 2D.1-2

Comparison of Peak Lake Levels

TIME OF PEAK		DIFFERENCES		RATIO
		TOL-CLE	CLE-BUF	T-C
DATE	HOUR	FT.	FT.	C-B
8	1000	1.2	1.5	0.80
8	1800	0.3	0.4	0.75
9	0100	0.6	0.8	0.75
9	0800	0.9	0.5	1.8
9	1600	2.8	2.3	1.2
9	2400	1.2	2.2	0.55
10	0800	0.9	0.7	1.3
Theory[1]		0.7	1.0	0.7

[1] Reference 9

TABLE 2D.1-3

Comparison of Peak Setup for Four Storms
Marblehead and Toledo

STORM	MEASURED PEAK SETUP (IGLD-1955 DATUM)		PEAK SETUP (WRT MONTHLY MEAN LAKE LEVEL)		RATIO
	TOLEDO	MARBLEHEAD	TOLEDO	MARBLEHEAD	MARBLEHEAD/ TOLEDO
April 26-28, 1966	573.07	571.76	2.76	1.54	0.56
	573.30[1]	572.74	2.99	2.52	0.84
Nov. 13-15, 1972	575.97	575.12	3.84	2.96	0.77
March 16-18, 1973	574.79	574.55	1.88	1.63	0.87
April 7-10, 1973	574.90	574.29	1.54	0.96	0.62
	575.47[1]	574.87	2.11	1.54	0.73

[1] Second Peak, corrected for cross-lake component.

2D.2.0 List Of References

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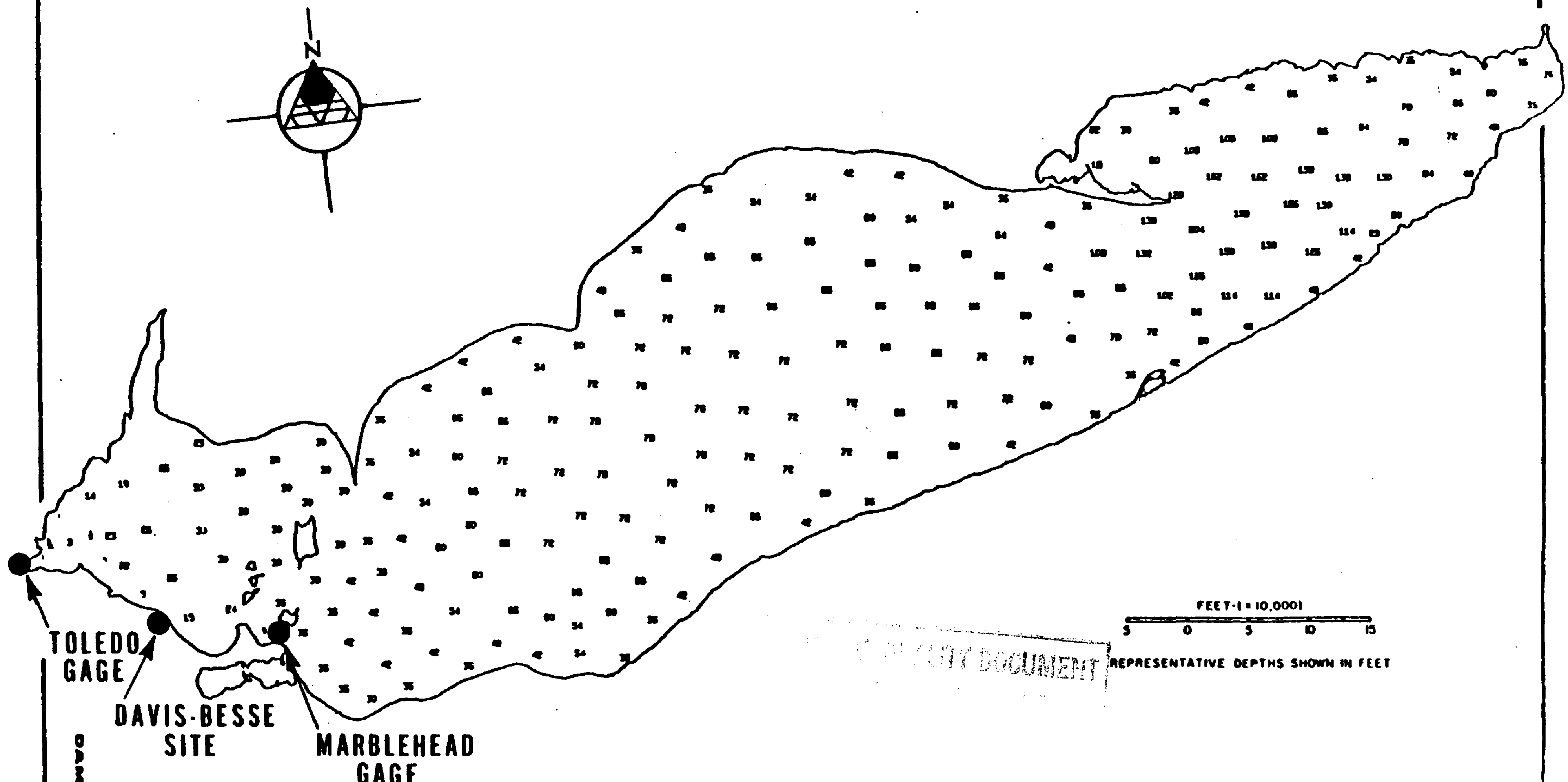

COMPUTER PLOT OF LAKE ERIE GEOMETRY & REPRESENTATIVE FIELD DEPTHS

REFERENCE: 9

DAMES & MOORE

FIGURE 2D.1-1
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JULY 1982

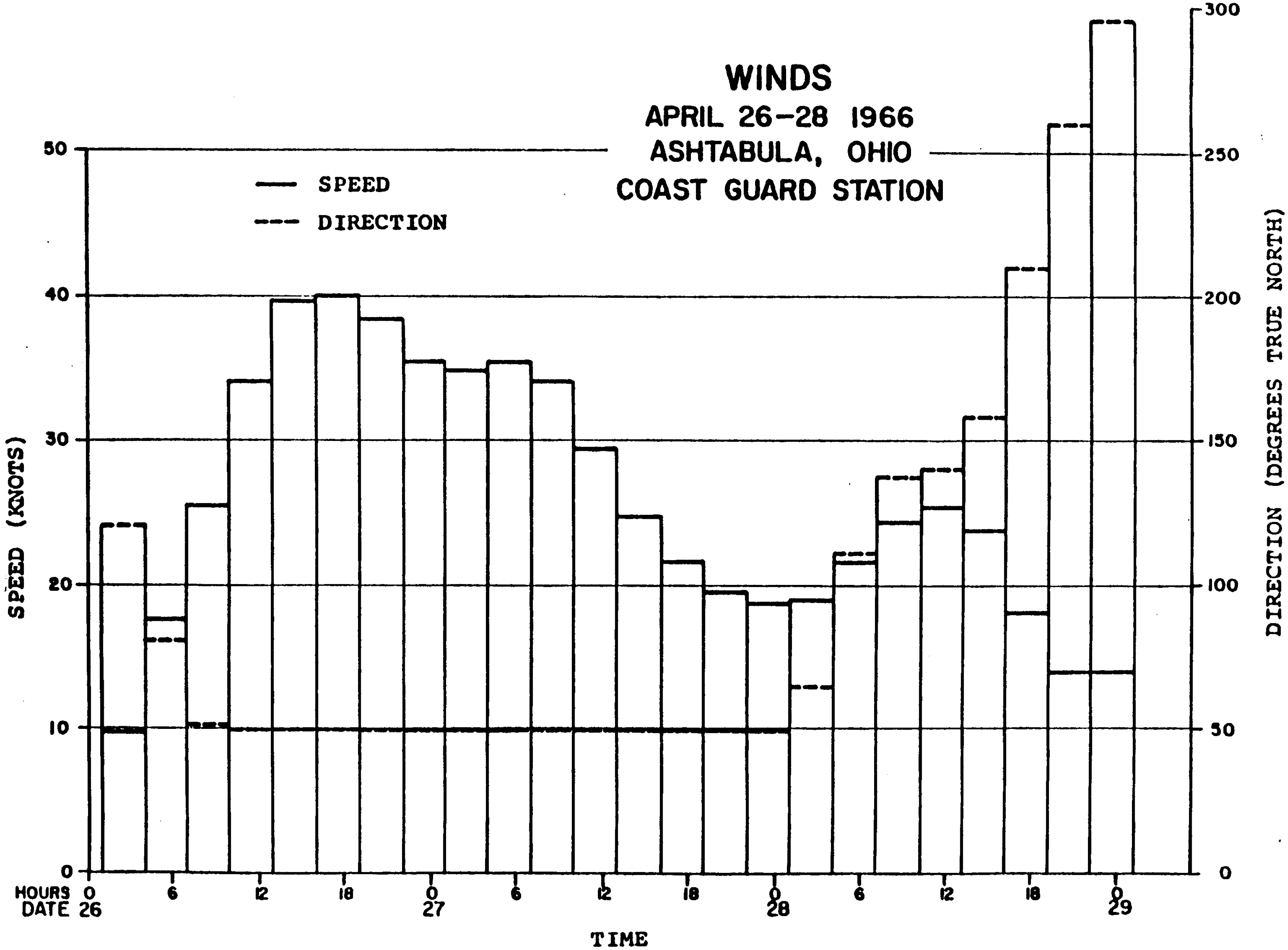

FIGURE 2D.1-2
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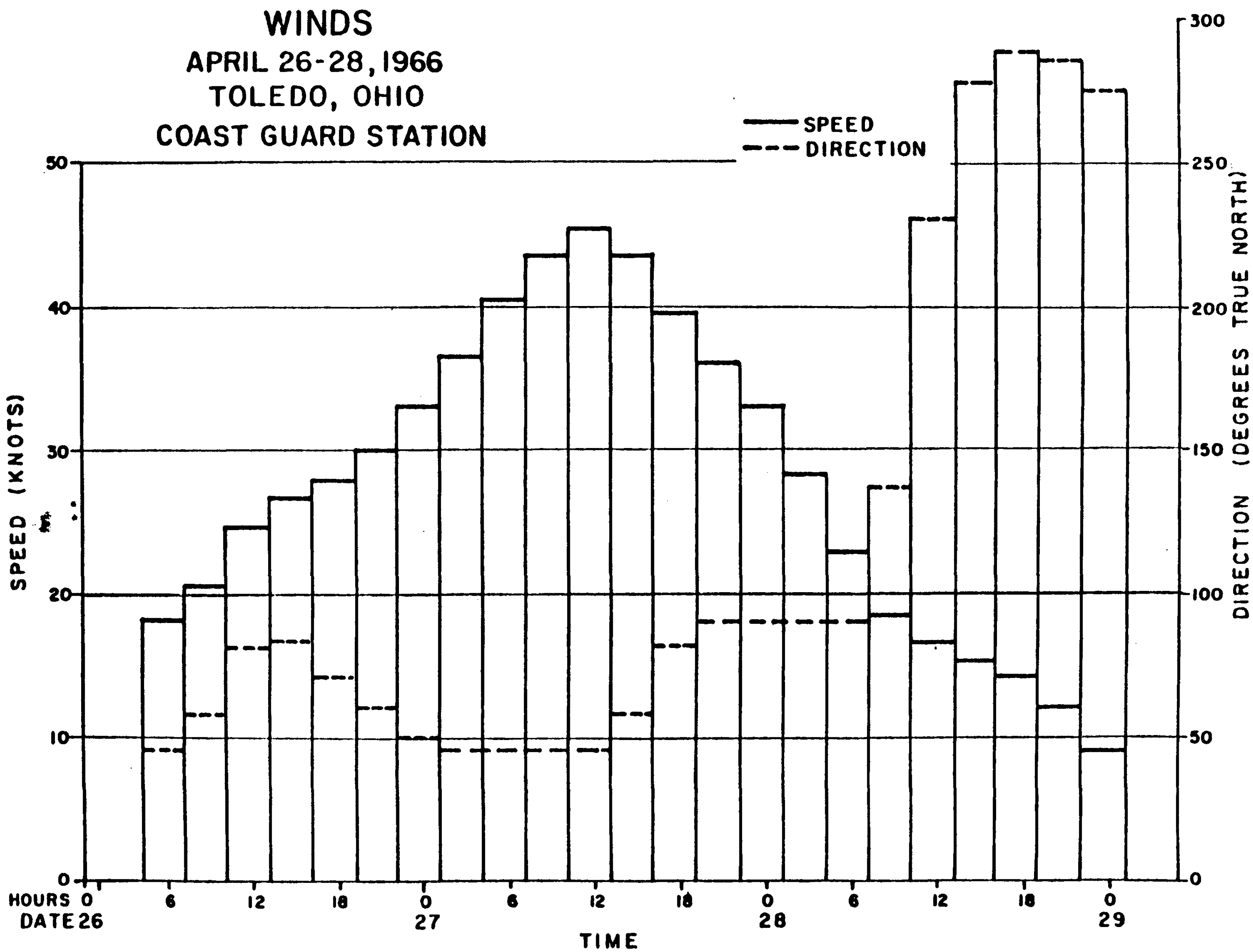

FIGURE 2D.1-3
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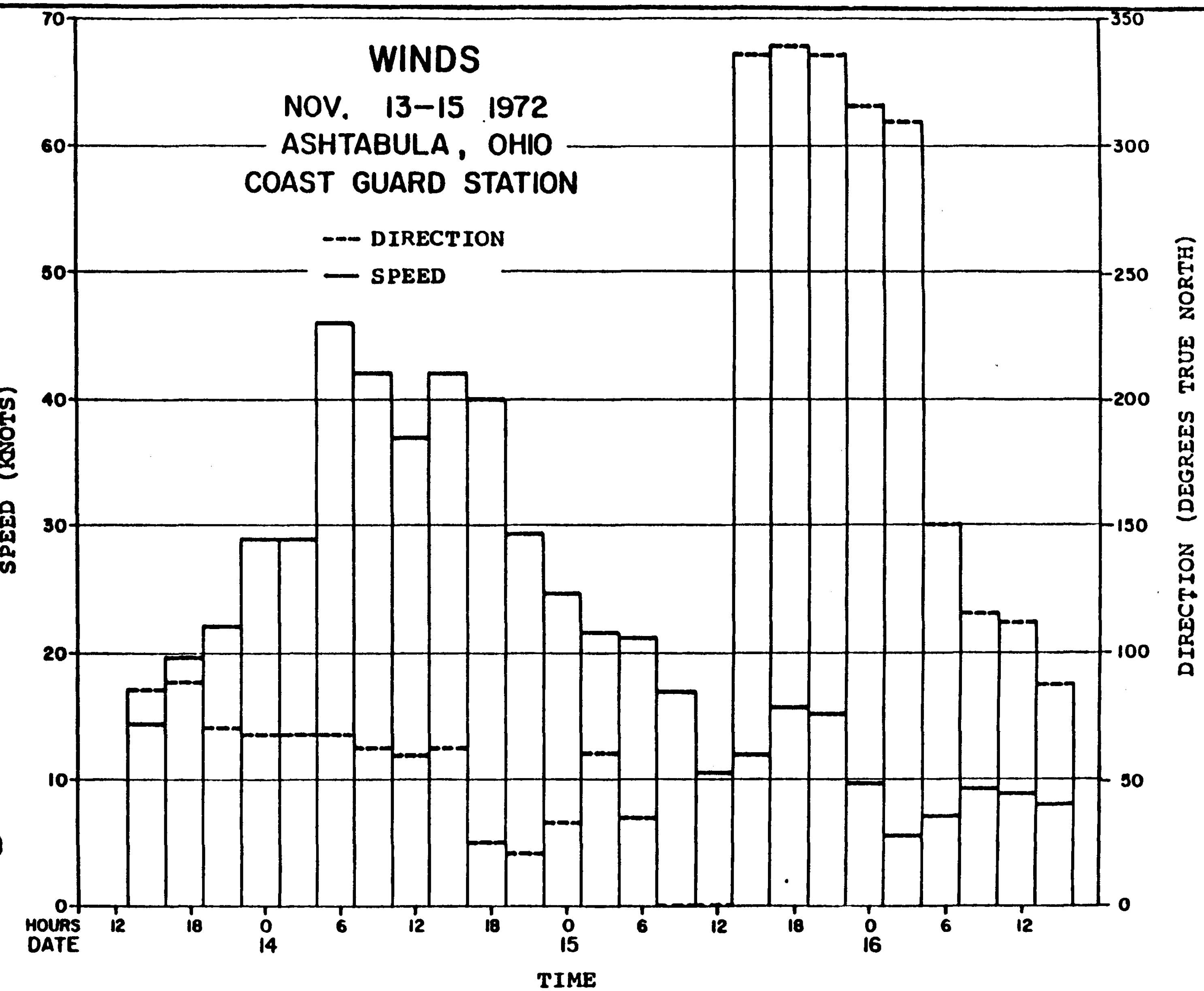

FIGURE 2D.1-4
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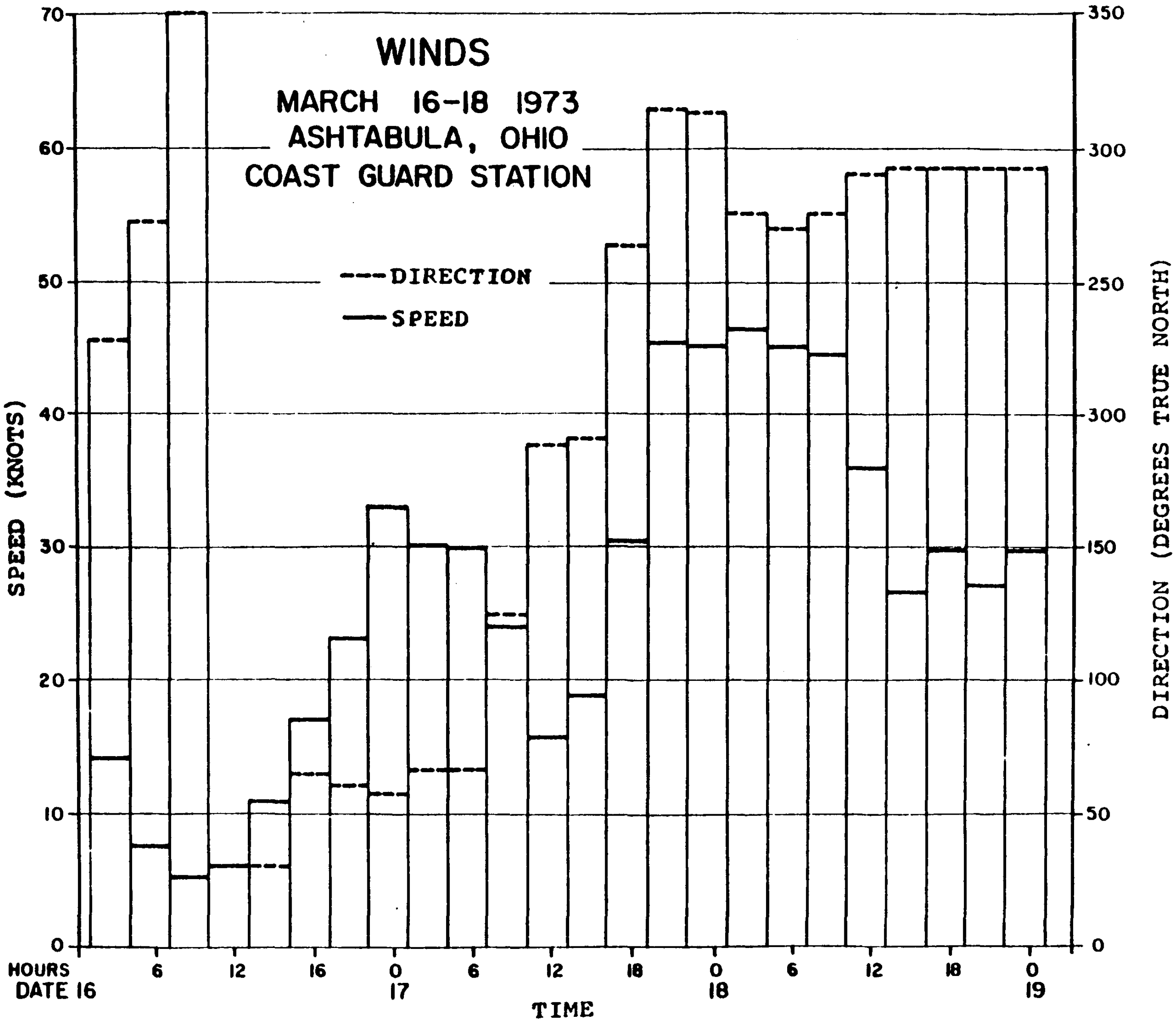

FIGURE 2D.1-5
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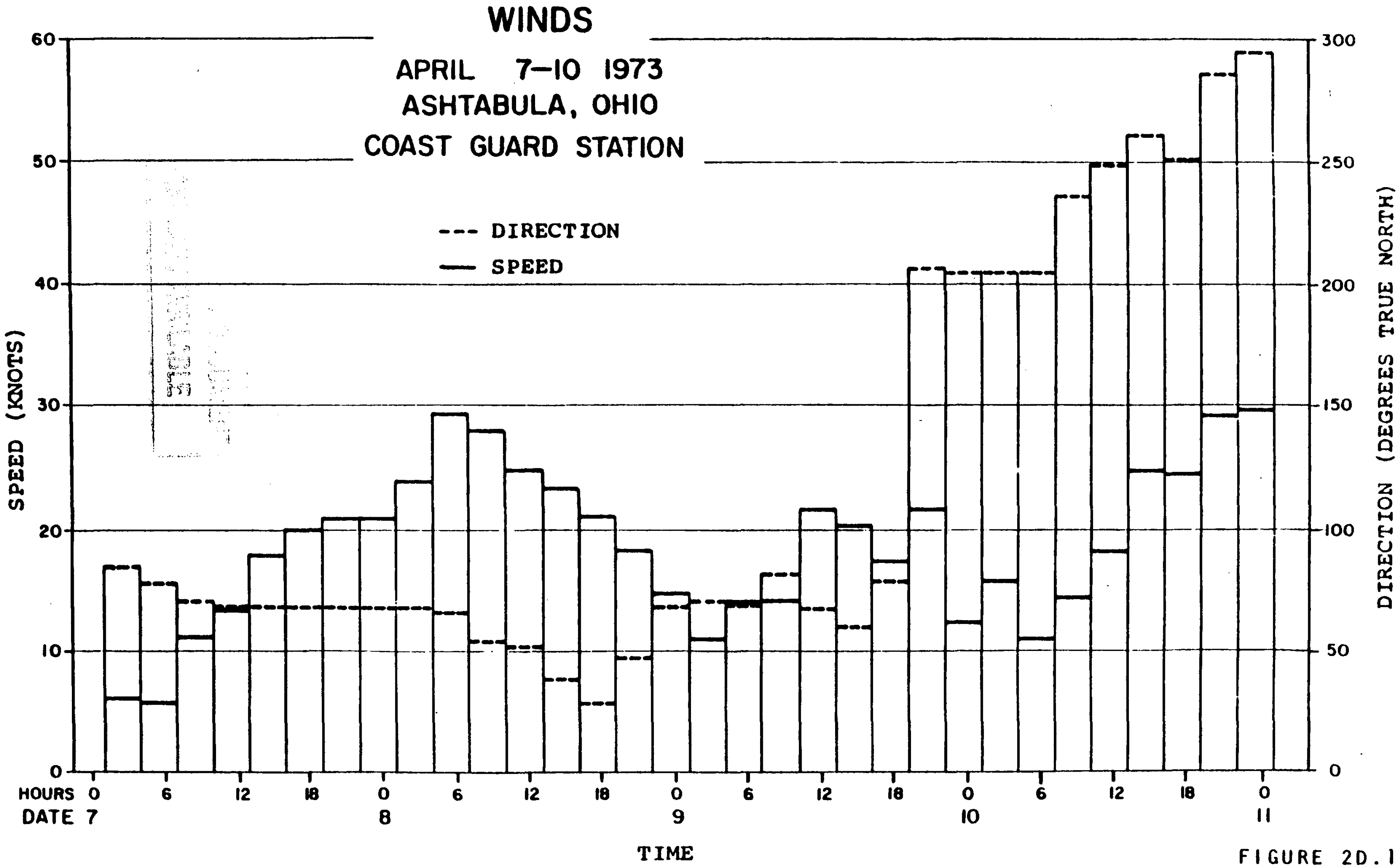

FIGURE 2D.1-6
REVISION 0
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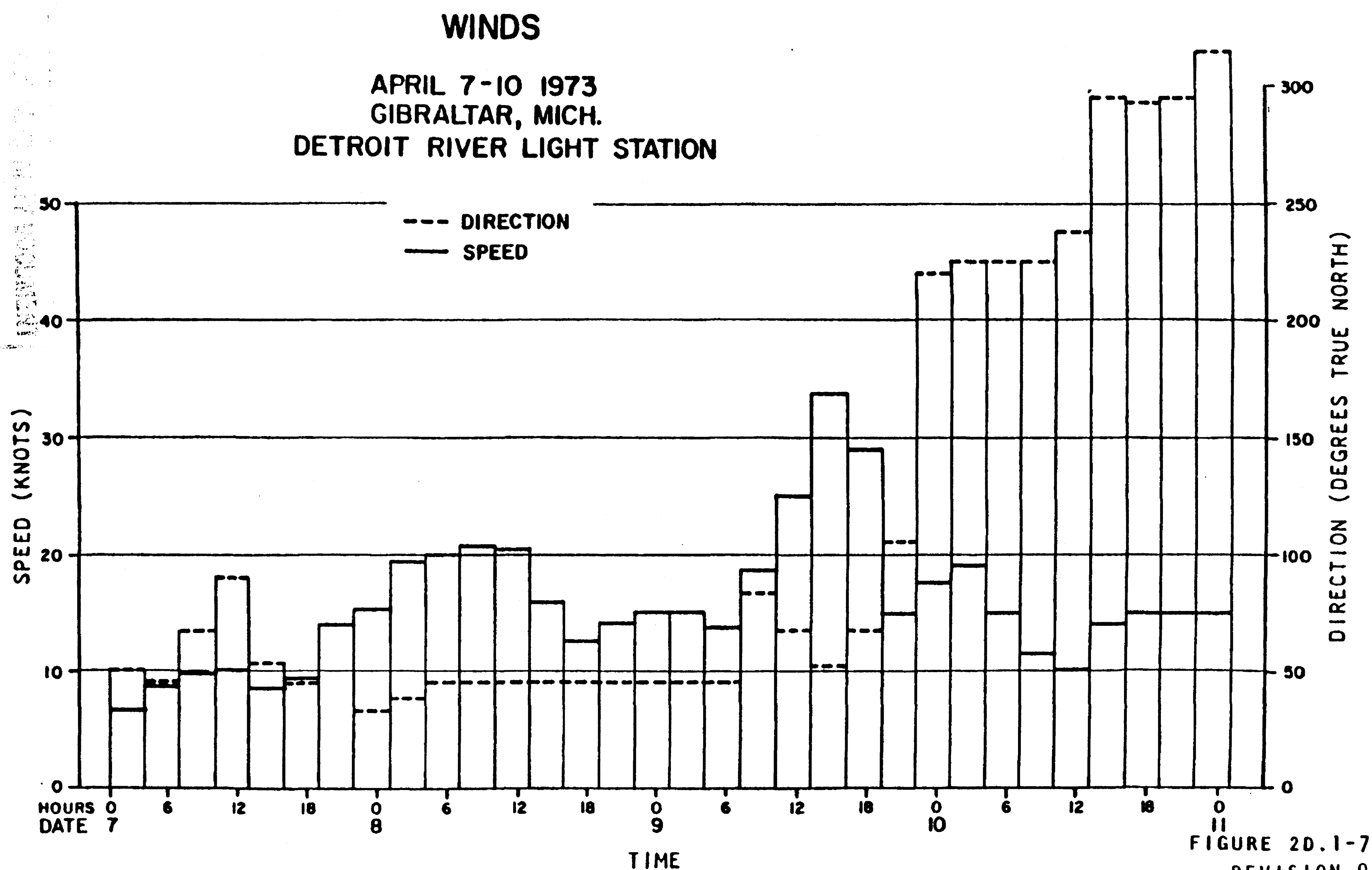

FIGURE 2D.1-7
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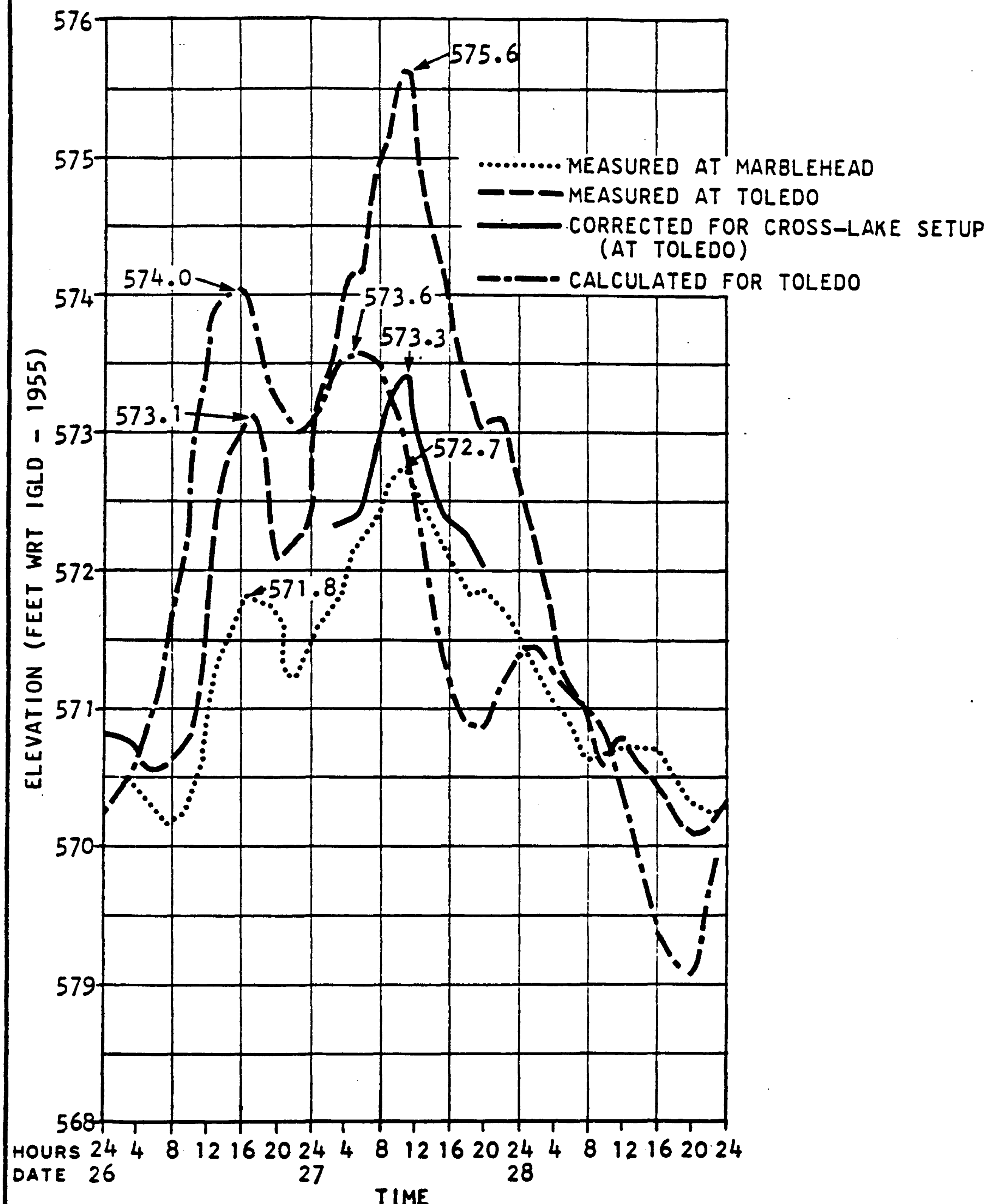
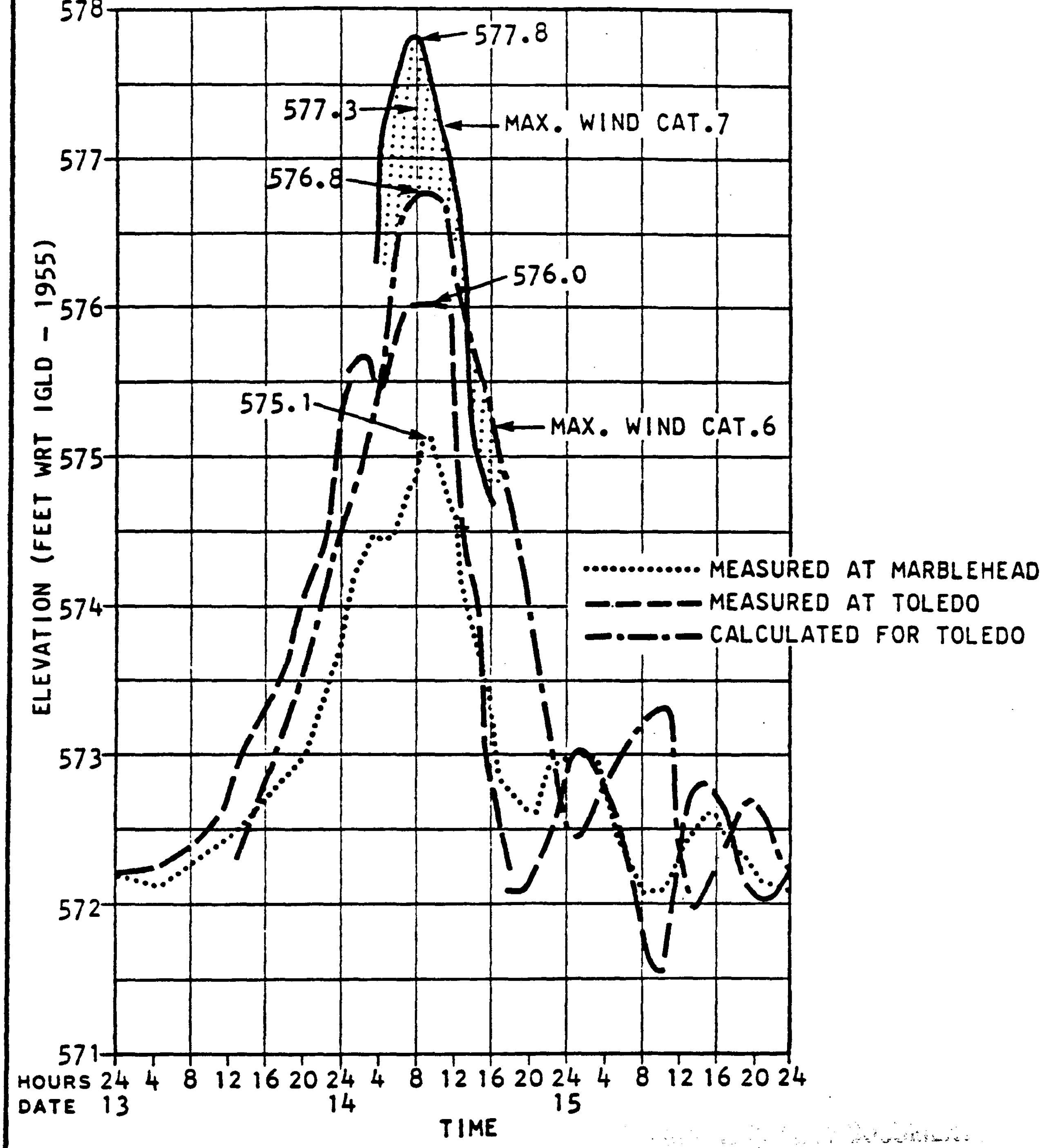


SETUP
APRIL 26-28 1966
TOLEDO, OHIO

FIGURE 2D.1-8

REVISION 0
JULY 1982



SETUP
NOVEMBER 13-15 1972
TOLEDO, OHIO

FIGURE 2D.1-9
REVISION 0
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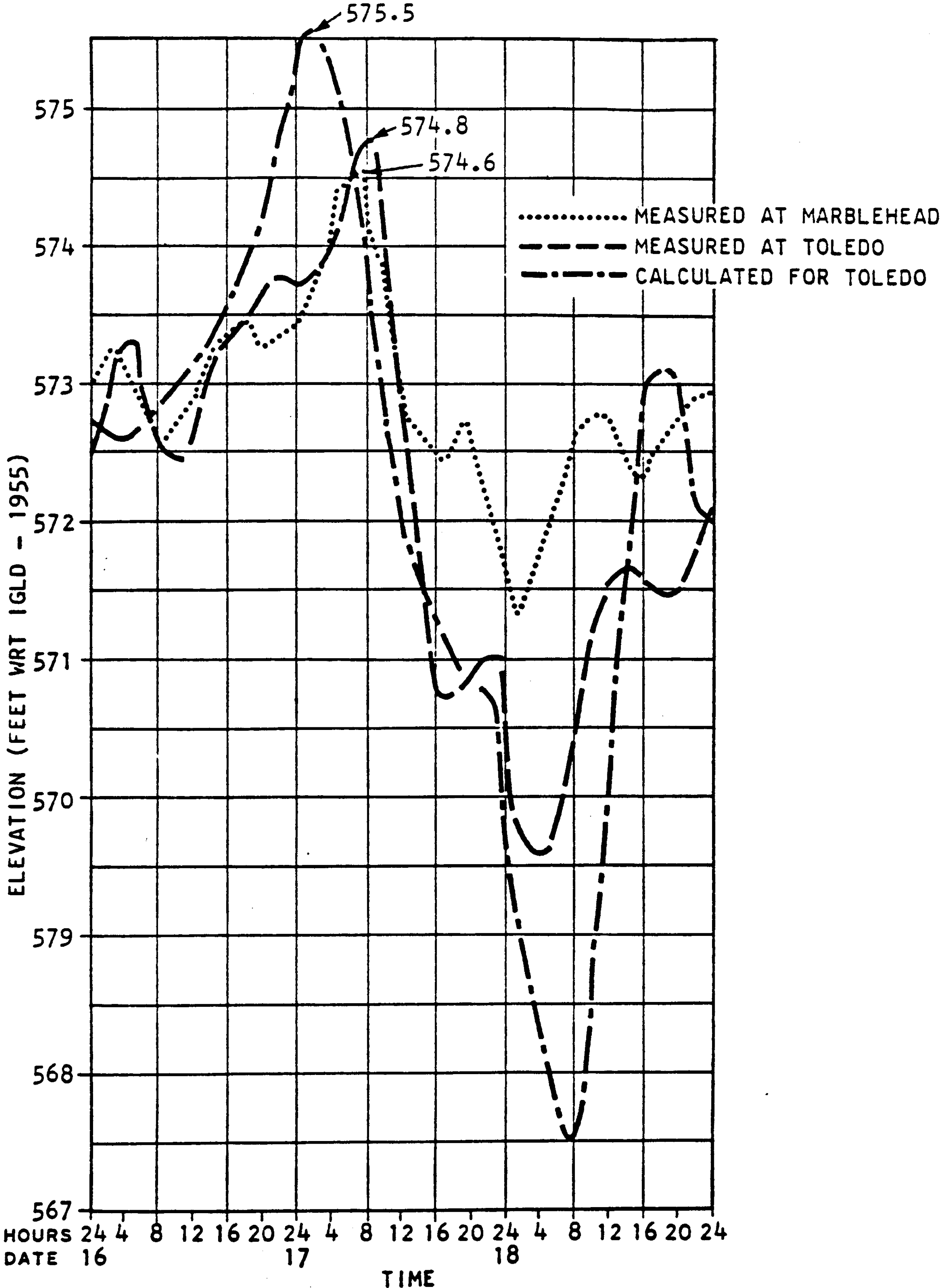
DAMES & MOORE



SETUP
MARCH 16-18 1973
TOLEDO, OHIO

FIGURE 2D.1-10
REVISION 0
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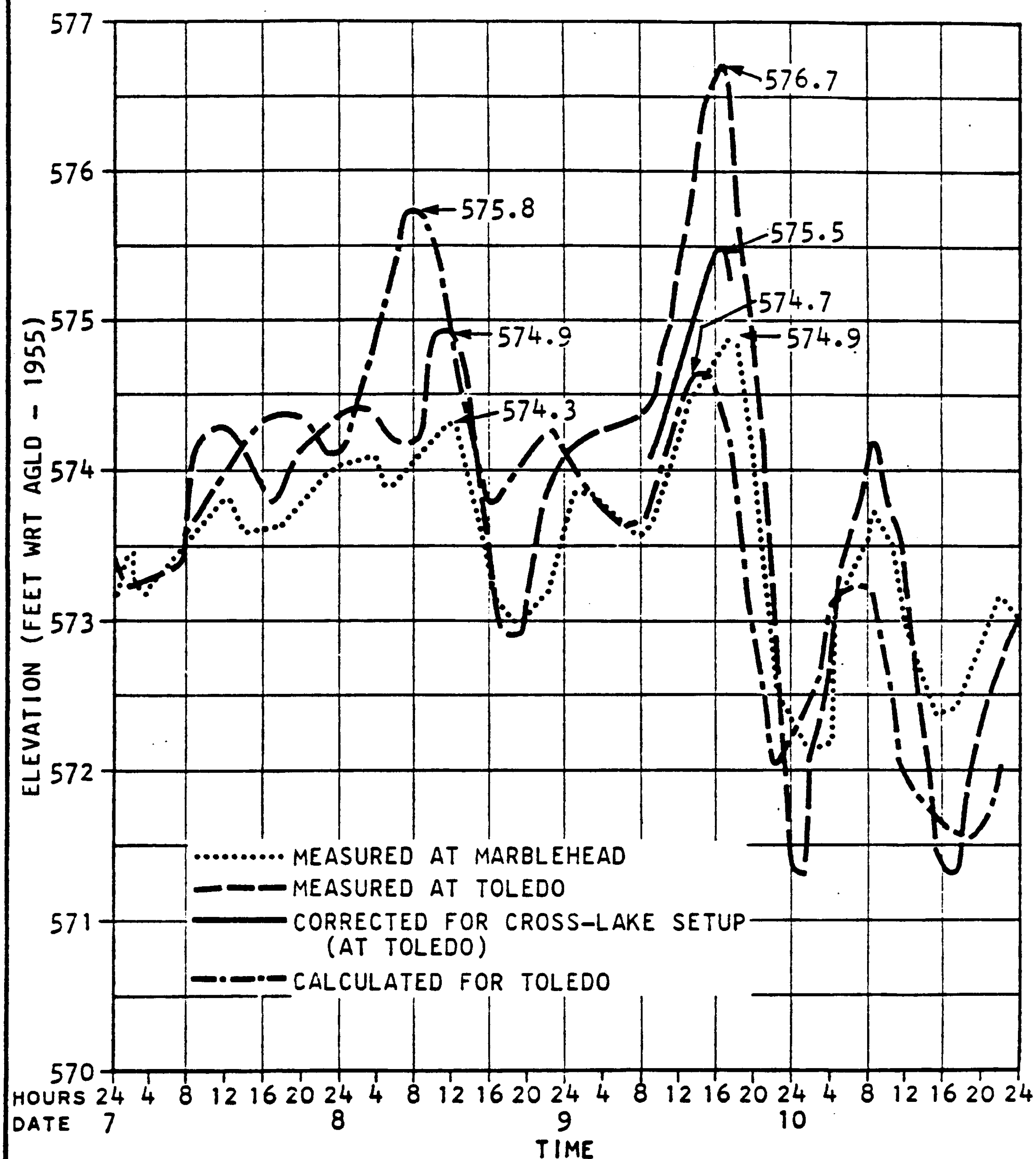
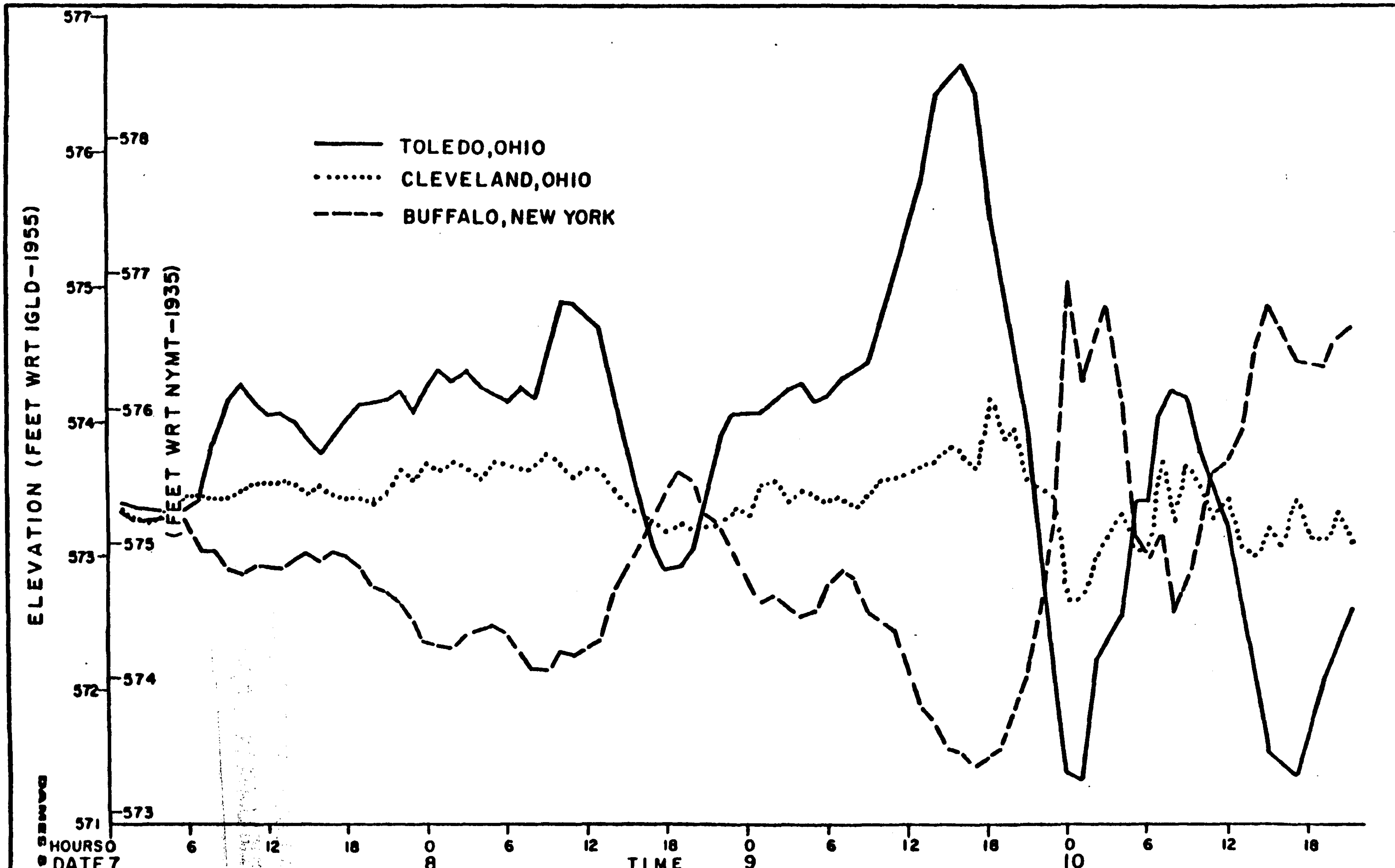


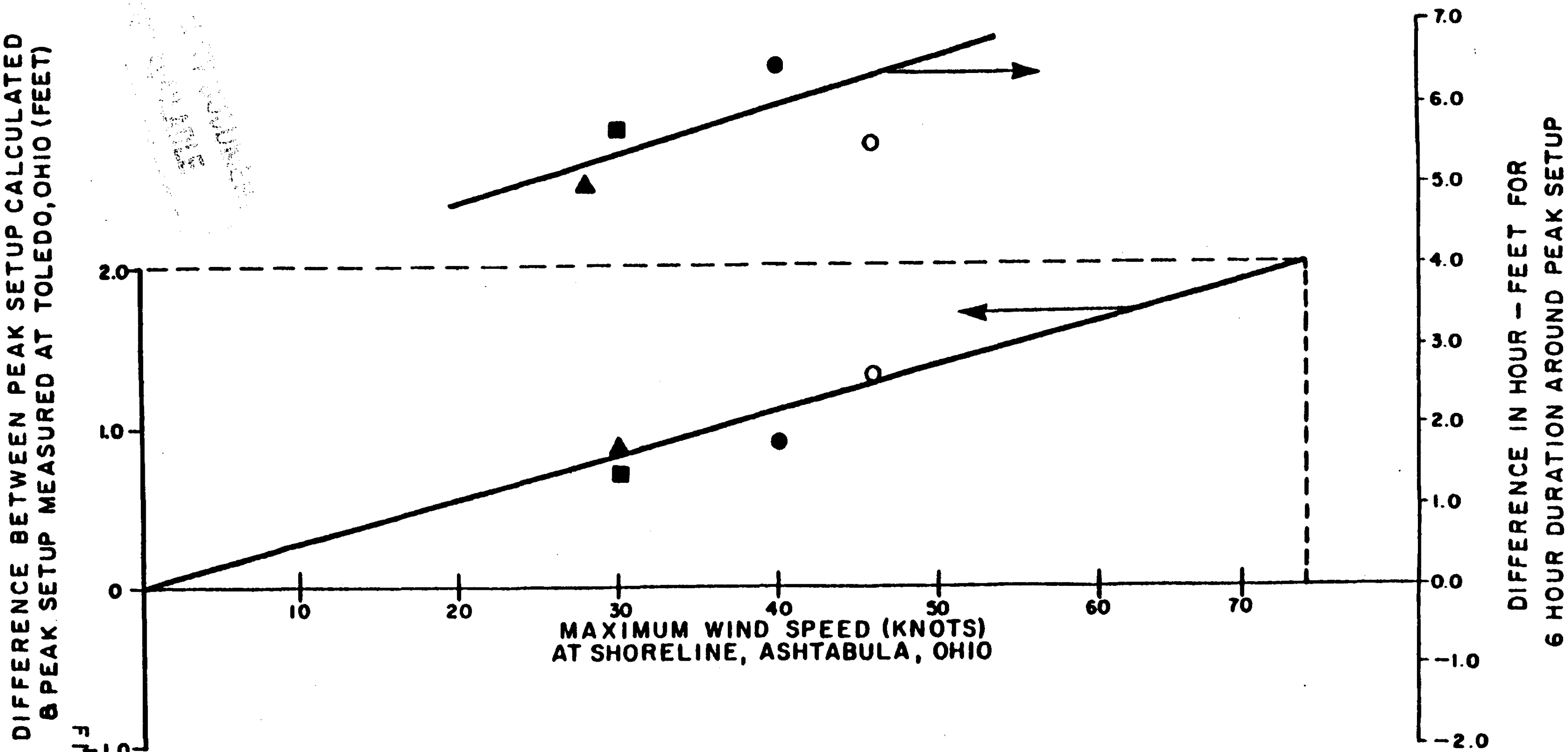
SETUP
APRIL 7-10 1973
TOLEDO, OHIO

FIGURE 2D.1-11
REVISION 0
JULY 1982
DAMES & MOORE



LAKE ERIE WATER LEVELS
APRIL 7-10 1973

FIGURE 2D.1-12
REVISION 6
JULY 1982



STORM

- ● APRIL 1966
- ○ NOV. 1972
- ■ MARCH 1973
- ▲ APRIL 1973

CALIBRATION OF PLATZMAN'S WIND SETUP MODEL

FIGURE 2D.1-13
REVISION 0
JULY 1982

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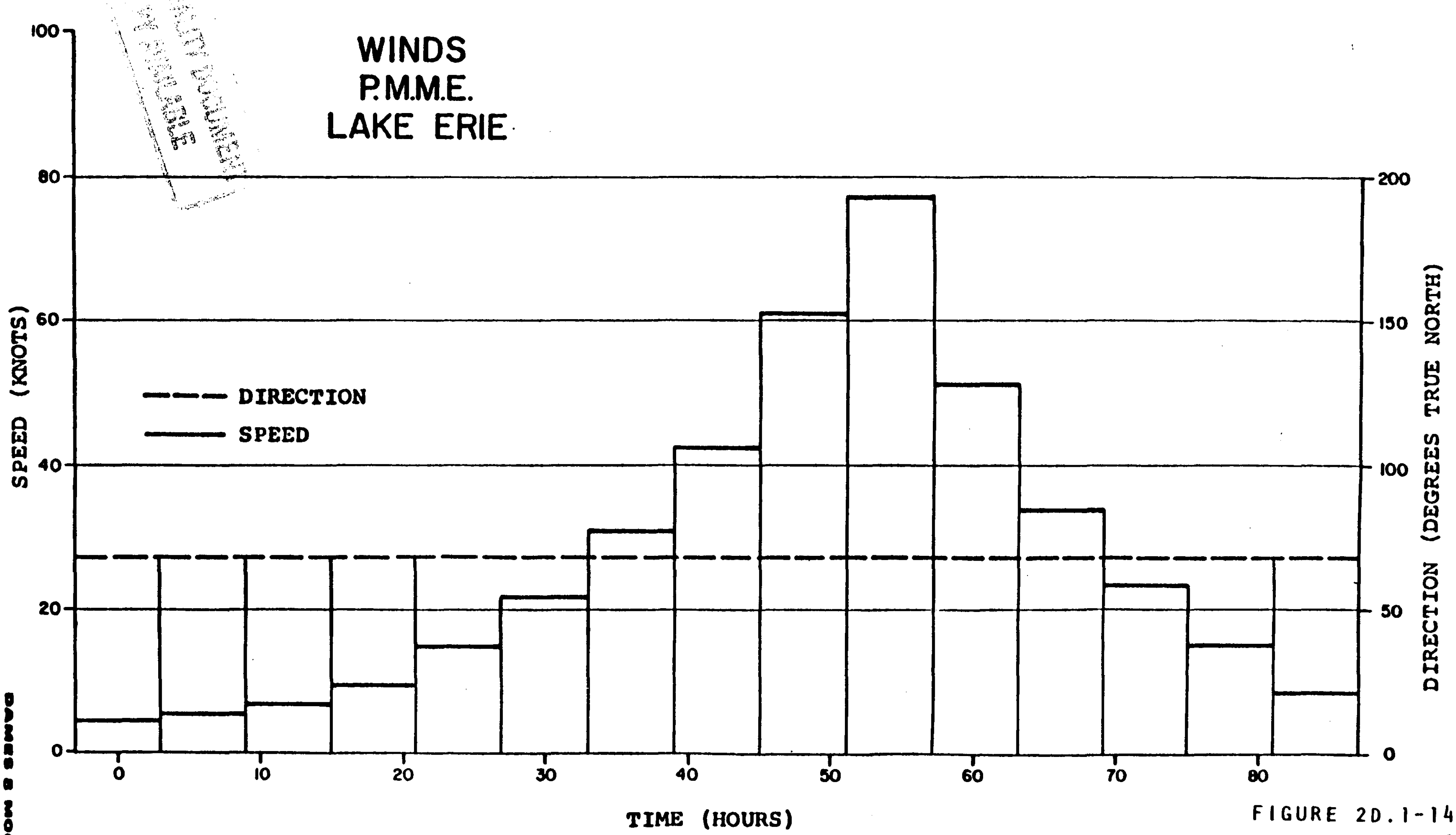

FIGURE 2D.1-14
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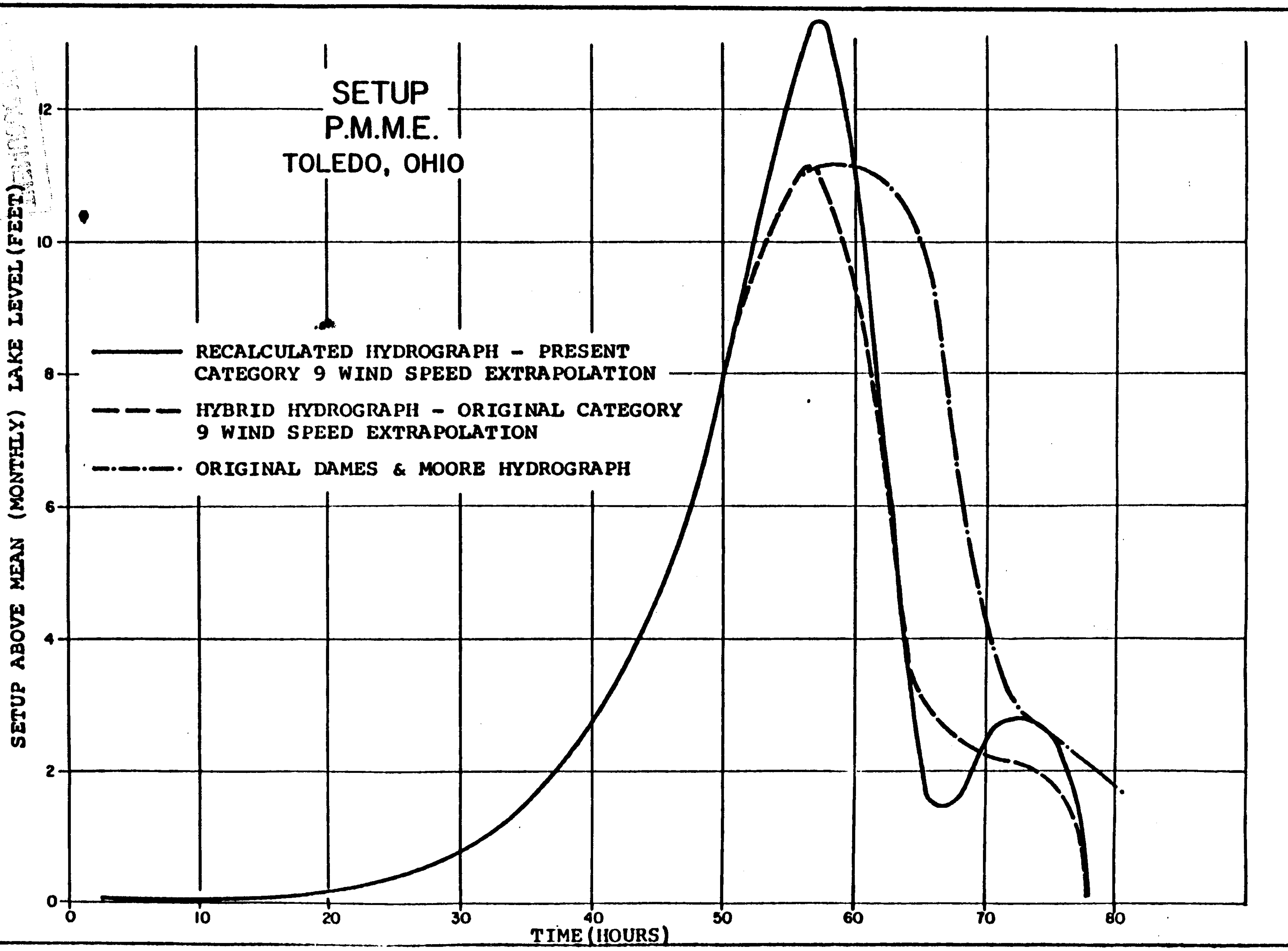
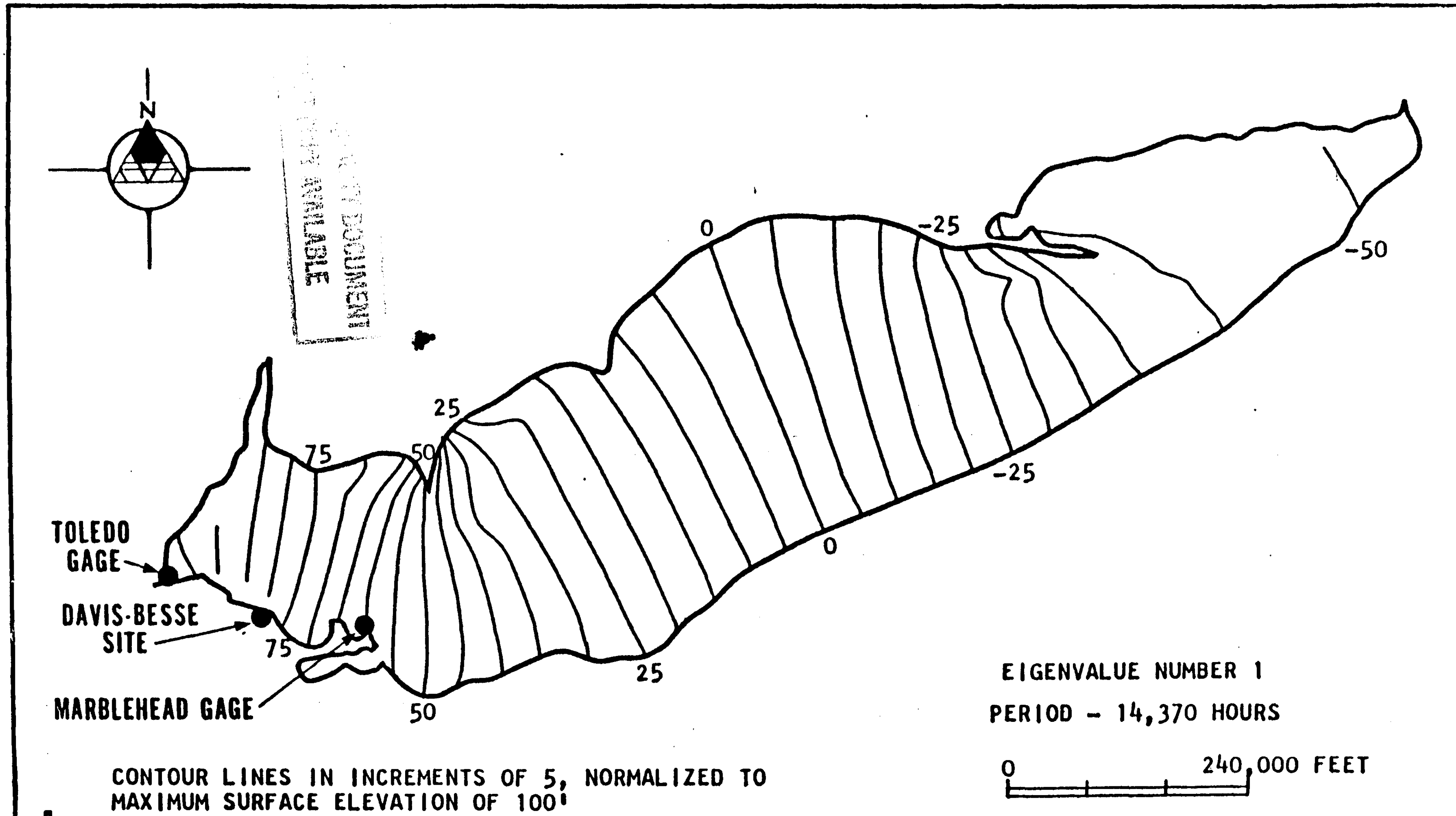


FIGURE 2D.1-15 REVISION 0 JULY 1982

DAMES & MOORE



MODE SHAPE - FIRST EIGENVALUE

FIGURE 2D.1-16

REVISION 0
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