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DRAWINGS CITED IN THIS APPENDIX*

*The listed drawings are included as "General References" only; i.e., refer to the drawings to obtain additional detail or to obtain background information. These drawings are not part of the UFSAR. They are controlled by the Controlled Documents Program.

DRAWING*	SUBJECT
A-4	South Elevation
M-5	General Arrangement Roof Plan Units 1 & 2
M-14	General Arrangement Section "A-A" Units 1 & 2
M-19	General Arrangement Lake Screen House Units 1 & 2

Attachments 2.5A through 2.5D have been deleted intentionally.

CHAPTER 2.0 - SITE CHARACTERISTICS

2.1 GEOGRAPHY AND DEMOGRAPHY

2.1.1 Site Location and Description

2.1.1.1 Specification of Location

Figure 2.1-1 shows the site within the state of Illinois, and Figure 2.1-2 outlines the site with respect to the Kankakee River and the county boundaries. The Braidwood site is located in Reed Township of Will County in northeastern Illinois approximately 50 miles southwest of Chicago and 20 miles south-southwest of Joliet. It is adjacent at its northwest corner to the village of Godley, and its western and southern borders lie adjacent to the Grundy County and Kankakee County boundary lines respectively. The site is in an area composed of flat agricultural farmland that has been scarred from coal strip mining. The site itself is located principally on terrain which has been stripped of this mineral resource.

At its closest point, the Kankakee River is approximately 3 miles east of the northeastern site boundary, which point is approximately 12 miles upstream from the headwaters of the Illinois River (confluence of the Kankakee and Des Plaines Rivers).

The coordinates of the center of the reactor containment buildings are given below in both latitude and longitude and Universal Transverse Mercator (UTM) coordinates. Latitude and longitude are given to the nearest second, and UTM coordinates are given to the nearest 100 meters.

UNIT	LATITUDE AND LONGITUDE	UTM COORDINATES
1	88° 13' 42" W x 41° 14' 38" N	4,565,300 N 397,000 E
2	88° 13' 42" W x 41° 14' 36" N	4,565,200 N 397,000 E

2.1.1.2 Site Area Map

The roughly rectangular site occupies approximately 4457 acres, of which 2537 acres comprise the main cooling pond. The pond will have an elevation of 595 feet above mean sea level (MSL) when filled to capacity. The plant property lines and the site boundary lines are the same except for the pipeline corridor.

The site boundary and the general outline of the pond are shown in Figure 2.1-3. As noted in this figure, the nuclear generating facilities are located at the northwest corner of the site. Figure 2.1-4 shows the location and orientation of principal plant structures. The makeup and blowdown lines are buried in the ground along a transmission line corridor. Their relation to the Kankakee River is shown in Figure 2.1-2.

The plant Exclusion Area Boundary (EAB) is also illustrated in Figure 2.1-5. The minimum exclusion boundary distance from the gaseous release point is 1625 feet.

There are no industrial, commercial, institutional, recreational or residential structures on the site. Illinois State Routes 53 and 129 are adjacent to the northwest boundary of the site. The Illinois Central Gulf Railroad (previously the Gulf, Mobile & Ohio Railroad) runs parallel between State Routes 53 and 129 and provides spur track access from the site area to the main line. Interstate 55 is less than 2 miles west-northwest of the site, and State Route 113 is approximately 2 miles north of the site. Figure 2.1-6 illustrates these transportation routes. The Kankakee River is approximately 3 miles east of the northeastern site boundary.

2.1.1.3 Boundaries for Establishing Effluent Release Limits

Title 10 of the Federal Code of Regulations Part 20.1302 requires that a licensee demonstrates by measurement or calculation that the total effective dose equivalent to the individual likely to receive the highest dose from the licensed operation does not exceed the annual dose limit.

10 CFR 50.34a also requires that "in the case of an application filed on or after January 2, 1971, the application shall also identify the design objectives, and the means to be employed, for keeping levels of radioactive material in effluents to unrestricted areas as low as practicable."

The unrestricted area boundary is the primary location used by the licensee in determining compliance with effluent release limits of the Radiological Effluent Technical Standards and the member of the public dose limit to 10CFR20. The unrestricted area is specified to be the site area boundary, or Braidwood Station property line. Expected concentrations of radionuclides in effluents are shown in Sections 11.2 and 11.3 and will be in compliance with the Radiological Effluent Technical Standards.

Figure 2.1-3 illustrates the Site Area Boundary, and Figure 2.1-2 shows the boundary with respect to the Kankakee River.

Distances from the release point of gaseous effluents (the vent stack) to the Site Area Boundary in the cardinal compass directions are given in Table 2.1-1. The site boundary closest to the release point of gaseous effluents (taken as the midpoint of the line drawn through the Unit 1 and Unit 2 station vent stacks) is in the northwesterly direction at a distance of 1625 feet.

Since liquid effluents are discharged into the cooling pond blowdown line which subsequently discharges into the Kankakee

River, any radionuclides in liquid effluents enter the unrestricted area at that point (the blowdown line outfall).

2.1.2 Exclusion Area Authority and Control

2.1.2.1 Authority

The Braidwood Exclusion Area is owned in fee simple and controlled by Exelon Generation Company. The Exclusion Area is within the site boundary as shown in Figure 2.1-5. All mineral rights and easements for the Exclusion Area are owned and maintained by Exelon Generation Company. As sole owner, Exelon Generation Company has authority to determine and control all activities in the Exclusion Area, including removal and exclusion of personnel or property from the site.

For accident releases, the minimum Exclusion Area Boundary (MEAB) is 485 meters in all directions, measured from the outer containment wall.

The value of 485 meters used in Chapter 15.0 accident assessments (see Table 15.0-14), is the shortest distance between the surface of the containment building and the EAB. Releases for a design basis loss-of-coolant accident are assumed to occur via this minimum distance pathway rather than via the vent stack. This assumption and the MEAB distances are consistent with methodology that was used in general practice prior to the issuance of Regulatory Guide 1.145 and is considered acceptable by the NRC.

2.1.2.2 Control of Activities Unrelated to Plant Operation

Exelon Generation Company retains the authority to control any and all activities on the plant site. The responsibility for implementing this authority lies with the plant supervisory staff. There is no one residing on the site, and only employees of Exelon Generation Company or other authorized personnel work on the site. Procedures have been established for controlling visitors to the plant.

2.1.2.3 Arrangements for Traffic Control

Since the Exclusion Area is not traversed by any highway, railway, or waterway, no traffic control arrangements are deemed necessary.

2.1.2.4 Abandonment or Relocation of Roads

Three township roads, two of which traverse the Exclusion Area, have been abandoned at the Braidwood Station. These abandoned roadways have no public access or usage and are under complete control of Exelon Generation Company.

All abandonment proceedings are complete. The Highway Commissioner of Reed Township has the authority possessed under state law to effect this abandonment. The following procedures were followed to achieve abandonment:

a. the Highway Commissioner of Reed Township was petitioned to close the roads,

b. public notice of hearing on this matter was given,

c. a public hearing was held, and

d. a final hearing was held and a final order issued.

Paul Abraham (Highway Commissioner of Reed Township) and Mildred Blecha (Town Clerk for Reed Township) were the public authorities who made the final determination. No roads will be relocated.

2.1.3 Population Distribution

The population projections and the list of cities with their projected populations, found in Tables 2.1-2, 2.1-3, 2.1-9, and 2.1-10 are generated by a system of Sargent & Lundy (S&L) developed computer programs (Reference 1). The demographic tables present the population figures broken into 16 directional segments and 10 distance increments surrounding the site, while the list of cities details populations in urban areas, their distance and direction from the site, and their 2020 projected populations.

The U.S. Bureau of the Census 1980 population for all townships between 10 and 50 miles of the station was proportioned into each of the 16 directional sectors and distance increments. The proportion of the population assigned to each sector was based on the proportion of land area of each township falling in that sector. In order to ensure that the figures more accurately represent the population distribution of an area, the proportioning technique incorporated knowledge of the area, location of outstanding features such as parks and military bases, and location of large populations in cities. The population thus derived from each sector was used as input to the computer program.

Projected population distributions were made by a computer program using a modified "ratio technique." The ratio technique essentially involves calculating the future population of an area by projecting the ratio of the total population of that area to the total population of a larger area containing the first, for which population projections have already been made. Projection of the ratio for this report included the following techniques: 1) the geographic units used for the ratio were state and township, 2) to determine the rate of change in the ratio for use in projection, the historical base period 1970 to 1980 was used, and 3) the rate of change in the ratio found during the base period was projected linearly for a few years, but was gradually decreased to zero--the ratio itself became constant after 20 years. The effect of the third technique is that the growth rate of the township may differ significantly from that of the state during the base period and for a few years thereafter, but after about 20 years the growth rates for the two areas will be the same. State projections required for use in the modified ratio technique were projected geometrically based on state growth during the base period.

For greater accuracy in the 0- to 10-mile region, a house count was obtained from a combination of data obtained from 1981 and 1982 aerial photographs, and field survey conducted in 1981. To estimate the population, the number of houses was multiplied by the average number of people per household in each township as listed in Table 2.1-11. These numbers are based on the number of housing units in the unincorporated areas of each township and the U.S. Census Bureau population statistics (Reference 2).

2.1.3.1 Population Within 10 Miles

The geographical location of the sectors within 10 miles are identified in Figure 2.1-7. Table 2.1-2 shows the 1980 and projected population distribution within 10 miles of the Braidwood Station. The total 1980 population is estimated at 27,482 with an average density of 87 persons per square mile within this area. The maximum population densities in the near vicinity of the station occur in the northern sectors, which includes the cities of Braidwood and Wilmington, and the village of Coal City.

Figure 2.1-8 shows the location of cities and villages within 10 miles and their 1980 population. Wilmington (1980 population 4,424), Braidwood (1980 population 3,429), and Coal City (1980 population 3,028) are the largest urbanized areas within 10 miles of the plant. The village of Godley (1980 population 373) located approximately 0.5 mile southwest of the station is the closest village.

The total population within 10 miles is projected to be 35,411 by 2020 with average density projected to be 113 within this region.

2.1.3.2 Population Between 10 and 50 Miles

The 1980 population distribution and the estimated projected population distribution through 2020 at 10-year intervals for the area between 10 and 50 miles are summarized in Table 2.1-3. The geographical locations of the population sectors are found in Figure 2.1-9. The total population within 50 miles was 4,580,641 in 1980 and is projected to approach 5,124,734 by 2020.

The most heavily populated sectors within 50 miles of the site lie in the north-northeast and northeast directions, with 1980 populations of 1,178,378 and 2,201,145 respectively. The high populations in these sectors are due primarily to the inclusion of the city of Joliet (1980 population 77,956) and a portion of Chicago (1980 population 3,005,072). Also included in this area are some suburbs of Chicago and cities in Lake County, Indiana.

The south and south-southwest sectors are the least populated sectors with an estimated population of 8,886 and 12,123 respectively.

2.1.3.3 Transient Population

The transient population within 10 miles of the site is composed of visitors to recreational facilities, students enrolled at and teaching staff employed by schools, and employees at industrial establishments.

As shown in Table 2.1-4, the state parks and conservation areas which are within a 10-mile radius of the site include the Des Plaines Conservation Area located approximately 8 miles north of the site, the Goose Lake Prairie State Park located approximately 9 miles north-northwest of the site, the Kankakee River State Park located approximately 9 miles east of the site, and the Illinois and Michigan Canal State Trail (Channahon Park Access) located approximately 10 miles north of the site. The total numbers of visitors to these areas during 1976 were 92,043, 60,728, 1,447,951, and 99,000 respectively. The estimated peak daily attendances for these areas are 1,000, 462, 33,000, and 1,000 visitors respectively.

The Des Plaines Conservation Area consists of 4253 acres and offers camping, picnicking, fishing, boating, and hunting (Reference 3). The Goose Lake Prairie State Park consists of 2357 acres, of which approximately 1513 acres are dedicated as an Illinois Nature Preserve. The park offers picnicking, hiking and a year round interpretive program (Reference 4). The Kankakee River State Park consists of 2968 acres extending along the Kankakee River and offers camping, picnicking, fishing, boating, hiking, horse trails, hunting and a summer interpretive program (Reference 5). The Illinois and Michigan Canal State Trail is currently being developed for hiking, bicycling and canoeing. The portion of the trail near the Channahon access is now completed and offers camping, canoeing, bicycling, and hiking (Reference 6).

In addition to these state recreational facilities, there are several privately owned recreation areas within 10 miles of the

Braidwood site. Table 2.1-4 lists these recreation areas along with their location, their total membership, and their estimated peak daily attendance. These clubs and parks provide a variety of recreational activities and attract people from outside the 10-mile radius.

The estimated peak daily attendance figures in Table 2.1-4 indicate that the population within 10 miles of the site could increase by 51,437 persons on a short-term basis due to both state and private facilities. Should all these visitors be from outside the 10-mile radius, the total population within the 10-mile area would increase by 233%.

As listed in Table 2.1-5, there are 10 industries within 10 miles of the site. Approximately 860 persons are employed at these industries. Even if all these people come from outside the 10-mile area, which is highly unlikely, the total population of this area would increase during working hours by only about 3%.

As shown in Table 2.1-6, there are 16 schools within 10 miles of the site, and they had a total 1981-1982 enrollment of 5625 students and a staff of 332 teachers. The great majority of students attending these schools reside within a 10-mile radius of the site.

The 1980 and projected population within the 10-mile radius from the site is given in Table 2.1-7. This table includes the residential population and the peak daily transient population resulting from recreational activities within the 10-mile area.

2.1.3.4 Low Population Zone

The Low Population Zone (LPZ) as defined in 10 CFR 100 is "the area immediately surrounding the Exclusion Area which contains residents, the total number and density of which are such that there is a reasonable probability that appropriate protection measures could be taken in their behalf in the event of a serious accident." 10 CFR 100.11 lists a numerical criterion to be met by the LPZ (for accidents analyzed using TID-14844), namely, that the LPZ be "of such size that an individual located at any point on its outer boundary who is exposed to the radioactive cloud resulting from the postulated fission product release (during the entire period of passage) would not receive a total radiation dose to the whole body in excess of 25 rem or a total radiation dose in excess of 300 rem to the thyroid from iodine exposure." For accidents analyzed using Regulatory Guide 1.183 (AST), dose limits (in Rem TEDE) are listed in 10 CFR 50.67.

The LPZ for the Braidwood Station is the area including the Exclusion Area within a 1-1/8-mile (1810-meter) radius (measured from the midpoint between the two reactors) of the site. This choice of the LPZ radius satisfies the radiation dose criteria (see Chapter 15.0). The closest population center of 25,000 persons or more is Joliet, Illinois, located approximately 20 miles north-northeast of the station.

Figure 2.1-10 depicts the highways, railroads, and recreational facilities within the LPZ. The 1980 and projected population within the LPZ by sectors is given in Table 2.1-8. This table includes the residential population and the transient populations resulting from activities in the LPZ.

As shown in Table 2.1-4, there is one private recreational facility located within the LPZ. The Chicago Beagle Club, located approximately 1/2 mile southwest of the site near the village of Godley, has 46 families who are members and an estimated peak daily attendance of 500 persons. Field trials are held three times a year (April, August, and November) for a duration of 1 day. A meeting to elect club officers is held in January. Some of the members (perhaps a dozen or less) also use the facilities on weekends for dog trials and training (Reference 7).

In addition to the recreational facility within the LPZ, there are 11 private recreational facilities located between 1-1/8 and 5 miles from the site. Their approximate locations, membership and estimated peak daily attendance are outlined in Table 2.1-4.

There are no schools within the LPZ. The nearest schools are the Braidwood Elementary School, located approximately 1.4 miles north-northeast in Braidwood, the Reed Custer High School, located approximately 1.4 miles north-northeast in Braidwood, and the Braceville Elementary School, located approximately 2.0 miles southwest in Braceville.

In addition to the above three schools, there are four schools located between 3 and 5 miles from the site. Table 2.1-6 outlines the approximate location and number of teachers and students for each school.

There are no industrial establishments within the LPZ. Table 2.1-5 outlines the industries within 5 miles of the site and gives their approximate number of employees. There are also no known commercial establishments located within the LPZ which could be expected to produce sizeable changes in the transient population of the area. The only known commercial establishment is the Hileman's Junk Yard, located approximately 2/3 mile north-northeast of the center of the reactors. The estimated 1980 and projected transient population within the LPZ is 500. This estimated transient population is related to the Chicago Beagle Club located within the LPZ.

2.1.3.5 Population Center

The nearest population center is Joliet, located approximately 20 miles north-northeast of the site. This distance meets the population center criterion of 10 CFR 100.11, namely, that a population center distance be "at least one and one-third times the distance from the reactor to the outer boundary of the low population zone." According to the 1980 population census, Joliet had a population of 77,956, a decrease of 3% during the last decade.

Kankakee, the second closest population center, located approximately 20 miles east-southeast, had a population of 30,141 in 1980. Joliet and Kankakee are projected to be 82,501 and 31,065, respectively, by 2020. Table 2.1-9 lists the population centers within 50 miles of the site with their 1980 and projected 2020 population, and Figure 2.1-11 locates them. There is a total of 25 population centers within a 50-mile radius. Most of these centers are located near the greater Chicago municipal area, 40 to 50 miles northeast of the site.

Table 2.1-10 shows the distance and approximate direction to and the 1980 population of all urban centers (population greater than 2500) within a 30-mile radius of the site along with their projected 2020 population. It should be noted that there are only 22 such urban centers and that only two of these, Joliet and Kankakee, are population centers.

2.1.3.6 Population Density

The average population density within 10 miles of the site is estimated to be 87 people/mi^2. The maximum densities within 10 miles are in and around the cities of Braidwood (1 to 3 miles north and north-northeast) and Wilmington (5-10 miles northeast and east-northeast). The population density within 10 miles is projected to be 113 people/mi^2 by 2020.

The average population density in 1980 within 50 miles of the site is estimated to be approximately 583 people/mi^2. By 2020, the average density is projected to reach 653 people/mi^2 within 50 miles. Figure 2.1-12 shows the 1980 and 2020 projected populations with relation to the uniform densities of 500 people/mi^2 and 1000 people/mi^2 respectively in each of the 16 compass directions within 50 miles of the plant site. Tables 2.1-2 and 2.1-3 detail the cumulative populations shown in Figure 2.1-12.

2.1.4 References

1. Demog Studies (DEMOG) 11.1.018-3.1 (1974), Sargent & Lundy Computer Program, revised 1982.

2. U.S. Department of Commerce, Bureau of the Census, 1980 Census of Population and Housing, Washington, D.C., 1981.

3. "Recreational Areas," Illinois Department of Conservation, State of Illinois, 1976.

4. "Goose Lake Prairie State Park," Illinois Department of Conservation, State of Illinois, 1974.

5. "Kankakee River State Park," Illinois Department of Conservation, State of Illinois, 1974.

6. "Illinois and Michigan Canal State Trail," Illinois Department of Conservation, State of Illinois, 1975.

7. Ms. P. Zidich, Co-President Chicago Beagle Club Telephone Conversation with Mr. B. Barickman, Commonwealth Edison, October 28, 1982.

TABLE 2.1-1

DISTANCE FROM GASEOUS EFFLUENT RELEASE POINT TO NEAREST SITE BOUNDARY IN THE CARDINAL COMPASS DIRECTIONS

DIRECTION	DISTANCE (ft)
N	2,000
NNE	3,000
NE	2,600
ENE	2,300
E	3,400
ESE	8,900
SE	11,200
SSE	11,300
S	15,200
SSW	3,200
SW	2,050
WSW	1,750
W	1,700
WNW	1,650
NW	1,625
NNW	1,675

TABLE 2.1-2

1980 AND PROJECTED POPULATION WITHIN 10 MILES OF THE BRAIDWOOD SITE

ESTIMATED 1980 POPULATION BY ANNUAL SECTORS

DISTANCE RANGE FROM SITE (MILES)

SECTOR	0.0 TO 1.0	1.0 TO 2.0	2.0 TO 3.0	3.0 TO 4.0	4.0 TO 5.0	5.0 TO 10.0	0.0 TO 10.0
N	34	690	389	15	2	309	1439
NNE	75	823	960	294	70	234	2456
NE	0	107	103	0	480	4735	5425
ENE	4	12	22	22	291	1980	2331
E	0	0	13	28	22	1027	1090
ESE	0	0	17	18	50	236	321
SE	0	0	4	9	8	156	177
SSE	0	0	60	9	235	358	662
S	0	0	0	3	3	686	692
SSW	0	8	17	29	173	849	1076
SW	402	296	214	19	89	1384	2404
WSW	82	218	188	37	26	163	714
W	0	34	179	3	11	794	1021
WNW	8	0	8	37	13	251	317
NW	4	25	42	1499	1340	928	3838
NNW	6	256	119	1692	526	920	3519
Sum for radial interval	615	2469	2335	3714	3339	15010	27482
Cummulative total to outer radius	615	3084	5419	9133	12472	---	27482
Average density (people/mi^2) in radial region	196	262	149	169	118	64	87

TABLE 2.1-2 (Cont'd)

PREDICTED 1990 POPULATION BY ANNUAL SECTORS

DISTANCE RANGE FROM SITE (MILES)

SECTOR	0.0 TO 1.0	1.0 TO 2.0	2.0 TO 3.0	3.0 TO 4.0	4.0 TO 5.0	5.0 TO 10.0	0.0 TO 10.0
N	44	890	502	18	2	356	1812
NNE	97	1061	1238	307	73	247	3023
NE	0	138	133	0	501	5037	5809
NE	5	15	26	25	327	2084	2482
E	0	0	15	31	25	1105	1176
ESE	0	0	20	20	56	269	365
SE	0	0	5	10	9	181	205
SSE	0	0	77	11	276	414	778
S	0	0	0	4	4	772	780
SSW	0	8	17	30	177	869	1101
SW	478	304	220	20	94	1473	2589
WSW	104	224	193	38	28	167	754
W	0	35	184	3	12	857	1091
WNW	8	0	8	38	14	297	365
NW	5	26	43	1560	1663	1291	4588
NNW	8	328	140	2246	715	1414	4851
Sum for Radial interval	749	3029	2821	4361	3976	16833	31769
Cummulative total to outer radius	749	3778	6599	10960	14936	---	31769
Average density (people/mi^2) in radial region	238	321	180	198	141	71	101

TABLE 2.1-2 (Cont'd)

PREDICTED 2000 POPULATION BY ANNUAL SECTORS

DISTANCE RANGE FROM SITE (MILES)

SECTOR	0.0 TO 1.0	1.0 TO 2.0	2.0 TO 3.0	3.0 TO 4.0	4.0 TO 5.0	5.0 TO 10.0	0.0 TO 10.0
N	47	956	539	19	2	375	1938
NNE	104	1140	1330	317	75	255	3221
NE	0	148	143	0	517	5219	6027
ENE	6	17	28	26	343	2154	2574
E	0	0	16	33	26	1148	1223
ESE	0	0	22	21	59	283	385
SE	0	0	6	11	10	191	218
SSE	0	0	83	12	291	436	822
S	0	0	0	4	4	809	817
SSW	0	8	18	31	181	893	1131
SW	506	313	226	20	97	1527	2689
WSW	111	230	199	39	29	171	779
W	0	36	189	3	12	891	1131
WNW	8	0	8	39	15	314	384
NW	6	26	44	1608	1776	1405	4865
NNW	8	352	148	2426	776	1561	5271
Sum for radial interval	796	3226	2999	4609	4213	17632	33475
Cummulative total to outer radius	796	4022	7021	11630	15843	---	33475
Average density (people/mi^2) in radial region	253	342	191	210	149	75	107

TABLE 2.1-2 (Cont'd)

PREDICTED 2010 POPULATION BY ANNUAL SECTORS

DISTANCE RANGE FROM SITE (MILES)

SECTOR	0.0 TO 1.0	1.0 TO 2.0	2.0 TO 3.0	3.0 TO 4.0	4.0 TO 5.0	5.0 TO 10.0	0.0 TO 10.0
N	48	983	554	20	2	386	1993
NNE	107	1173	1368	326	77	262	3313
NE	0	152	147	0	532	5368	6199
ENE	6	17	29	27	352	2216	2647
E	0	0	17	34	27	1180	1258
ESE	0	0	22	22	61	291	396
SE	0	0	6	11	10	197	224
SSE	0	0	86	12	299	448	845
S	0	0	0	4	4	832	840
SSW	0	9	18	31	186	919	1163
SW	520	322	233	21	100	1570	2766
WSW	114	237	204	40	30	176	801
W	0	37	195	3	13	917	1165
WNW	9	0	9	40	15	323	396
NW	6	27	46	1654	1826	1445	5004
NNW	9	362	153	2495	798	1605	5422
Sum for radial interval	819	3319	3087	4740	4332	18135	34432
Cummulative total to outer radius	819	4138	7225	11975	16297	---	34432
Average density (people/mi^2) in radial region	261	352	197	216	153	77	110

TABLE 2.1-2 (Cont'd)

PREDICTED 2020 POPULATION BY ANNUAL SECTORS

DISTANCE RANGE FROM SITE (MILES)

SECTOR	0.0 TO 1.0	1.0 TO 2.0	2.0 TO 3.0	3.0 TO 4.0	4.0 TO 5.0	5.0 TO 10.0	0.0 TO 10.0
N	50	1011	570	20	2	397	2050
NNE	110	1206	1407	336	80	270	3409
NE	0	157	151	0	547	5520	6375
ENE	6	18	30	27	363	2279	2723
E	0	0	17	35	27	1214	1293
ESE	0	0	23	22	62	299	406
SE	0	0	6	11	10	202	229
SSE	0	0	88	12	308	461	869
S	0	0	0	4	4	856	864
SSW	0	9	19	32	192	945	1197
SW	535	331	239	21	103	1615	2844
WSW	117	244	210	42	31	181	825
W	0	38	200	4	13	943	1198
WNW	9	0	9	42	16	332	408
NW	6	28	47	1701	1878	1486	5146
NNW	9	372	157	2566	820	1651	5575
Sum for Radial interval	842	3414	3173	4875	4456	18651	35411
Cummulative total to outer radius	842	4256	7429	12304	16760	---	35411
Average density (people/mi^2) in radial region	268	362	202	222	158	79	113

TABLE 2.1-3

1980 AND PROJECTED POPULATION DISTRIBUTION WITHIN 10 - 50 MILES OF THE BRAIDWOOD SITE

ESTIMATED 1980 POPULATION BY ANNULAR SECTORS

DISTANCE RANGE FROM SITE (MILES)

SECTOR	10.0 TO 20.0	20.0 TO 30.0	30.0 TO 40.0	40.0 TO 50.0	10.0 TO 50.0	0.0 TO 50.0
N	18118	21607	159852	196880	396457	397896
NNE	18014	140555	210493	806860	1175922	1178378
NE	4170	31037	328860	1831653	2195720	2201145
ENE	1252	7008	135725	251879	395864	398195
E	1875	7055	6972	16999	32901	33991
ESE	25876	45742	9524	3854	84996	85317
SE	3479	6320	2591	9739	22129	22306
SSE	1963	1977	5545	2618	12103	12765
S	1191	1583	2918	2502	8194	8886
SSW	833	1395	6401	2418	11047	12123
SW	4926	2012	14651	6144	27733	30137
WSW	711	2612	21515	5561	30399	31113
W	1075	2013	8987	31459	43534	44555
WNW	1970	9491	19687	4206	35354	35671
NW	11138	3675	12042	4979	31834	35672
NNW	1840	6195	29119	11818	48972	52491
Sum for radial interval	98431	290277	974882	3189569	4553159	4580641
Cummulative total to outer radius	125913	416190	1391072	4580641	---	4580641
Average density (people/mi^2) in radial region	104	185	443	1128	604	583

TABLE 2.1-3 (Cont'd)

PREDICTED 1990 POPULATION BY ANNULAR SECTORS

DISTANCE RANGE FROM SITE (MILES)

SECTOR	10.0 TO 20.0	20.0 TO 30.0	30.0 TO 40.0	40.0 TO 50.0	10.0 TO 50.0	0.0 TO 50.0
N	24174	27526	187765	247373	486838	488650
NNE	18675	150493	268507	790971	1228646	1231669
NE	5273	44293	379601	1676391	2105558	2111367
ENE	1367	8580	154612	258285	422844	425326
E	1219	5192	8140	22523	37074	38250
ESE	30443	47173	10307	4231	92154	92519
SE	3821	6630	2636	9663	22750	22955
SSE	2140	2005	5524	2388	12057	12835
S	1313	1577	2730	2247	7867	8647
SSW	849	1368	6436	2083	10736	11837
SW	5268	1926	15657	6067	28918	31507
WSW	622	2499	20820	5292	29233	29987
W	1087	2349	9290	30514	43240	44331
WNW	2246	10293	18757	4296	35592	35957
NW	11881	4141	14168	4814	35004	39592
NNW	2127	7868	34744	14888	59627	64478
Sum for radial interval	112505	323913	1139694	3082026	4658138	4689907
Cummulative total to outer radius	144274	468187	1607881	4689907	---	4689907
Average density (people/mi^2) in radial region	119	206	518	1090	618	597

TABLE 2.1-3 (Cont'd)

PREDICTED 2000 POPULATION BY ANNULAR SECTORS

DISTANCE RANGE FROM SITE (MILES)

SECTOR	10.0 TO 20.0	20.0 TO 30.0	30.0 TO 40.0	40.0 TO 50.0	10.0 TO 50.0	0.0 TO 50.0
N	26127	29520	198444	264664	518755	520693
NNE	19241	156133	288030	804612	1268016	1271237
NE	5648	48379	399855	1678125	2132007	2138034
ENE	1424	9137	162464	270509	443534	446108
E	1092	4868	8640	24871	39471	40694
ESE	32184	48546	10729	4506	95965	96350
SE	3985	6849	2704	9862	23400	23618
SSE	2228	2056	5640	2387	12311	13133
S	1370	1610	2746	2236	7962	8779
SSW	871	1392	6585	2051	10899	12030
SW	5464	1948	16237	6182	29831	32520
WSW	615	2527	21114	5345	29601	30380
W	1114	2480	9565	30962	44121	45252
WNW	2360	10708	18950	4412	36430	36814
NW	12317	4342	14978	4881	36518	41383
NNW	2241	8433	36828	15936	63438	68709
Sum for radial interval	118281	338928	1203509	3131541	4792259	4825734
Cummulative total to outer radius	151756	490684	1694193	4825734	---	4825734
Average density (people/mi^2) in radial region	126	216	547	1108	636	614

TABLE 2.1-3 (Cont'd)

PREDICTED 2010 POPULATION BY ANNULAR SECTORS

DISTANCE RANGE FROM SITE (MILES)

SECTOR	10.0 TO 20.0	20.0 TO 30.0	30.0 TO 40.0	40.0 TO 50.0	10.0 TO 50.0	0.0 TO 50.0
N	26871	30361	204093	272197	533522	535515
NNE	19788	160577	296228	827514	1304107	1307420
NE	5809	49756	411236	1727849	2194650	2200849
ENE	1465	9397	167114	285085	463061	465708
E	1123	5007	8948	26282	41360	42618
ESE	33100	49928	11049	4758	98835	99231
SE	4098	7043	2781	10150	24072	24296
SSE	2292	2114	5801	2455	12662	13507
S	1409	1656	2824	2300	8189	9029
SSW	896	1431	6772	2109	11208	12371
SW	5619	2004	16699	6358	30680	33446
WSW	632	2599	21715	5497	30443	31244
W	1146	2550	9837	31843	45376	46541
WNW	2427	11013	19490	4538	37468	37864
NW	12668	4465	15404	5019	37556	42560
NNW	2304	8673	37876	16389	65242	70664
Sum for radial interval	121647	348574	1237867	3230343	4938431	4972863
Cummulative total to Outer radius	156079	504653	1742520	4972863	---	4972863
Average density (people/mi^2) in radial region	129	222	563	1143	655	633

TABLE 2.1-3 (Cont'd)

PREDICTED 2020 POPULATION BY ANNULAR SECTORS

DISTANCE RANGE FROM SITE (MILES)

SECTOR	10.0 TO 20.0	20.0 TO 30.0	30.0 TO 40.0	40.0 TO 50.0	10.0 TO 50.0	0.0 TO 50.0
N	27636	31225	209902	279945	548708	550758
NNE	20352	165147	304659	851067	1341225	1344634
NE	5974	51172	422941	1779099	2259186	2265561
ENE	1507	9665	171899	300466	483537	486260
E	1155	5149	9267	27773	43344	44637
ESE	34042	51349	11379	5025	101795	102201
SE	4215	7244	2860	10447	24766	24995
SSE	2357	2175	5966	2525	13023	13892
S	1449	1703	2905	2365	8422	9286
SSW	921	1472	6965	2169	11527	12724
SW	5779	2061	17175	6539	31554	34398
WSW	650	2673	22333	5654	31310	32135
W	1178	2623	10117	32749	46667	47865
WNW	2497	11326	20044	4667	38534	38942
NW	13028	4592	15843	5162	38625	43771
NNW	2370	8920	38954	16856	67100	72675
Sum for radial interval	125110	358496	1273209	3332508	5089323	5124734
Cummulative total to outer radius	160521	519017	1792226	5124734	---	5124734
Average density (people/mi^2) in radial region	133	228	579	1179	675	653

TABLE 2.1-4

RECREATIONAL FACILITIES WITHIN 10 MILES OF THE SITE

I. STATE FACILITIES

FACILITY	APPROXIMATE DISTANCE AND DIRECTION FROM THE SITE (mi)	1976 TOTAL[(1)] ATTENDANCE	ESTIMATED PEAK DAILY ATTENDANCE
Des Plaines Conservation Area	8 N	92,043	1,000[(2)]
Goose Lake Prairie State Park	9 NNW	60,728	462[(3)]
Kankakee River State Park	9 E	1,447,951	33,000[(4)]
Illinois and Michigan (Canal State Trail Channahon Park Access)	10 N	99,000	800-1,000[(5)]

II. PRIVATE PARKS AND CLUBS

FACILITY	APPROXIMATE DISTANCE AND DIRECTION FROM THE SITE (mi)	TOTAL MEMBERSHIP, FAMILIES	ESTIMATED PEAK DAILY ATTENDANCE, PERSONS
Chicago Beagle Club[(6)]	.5 SW	46	500
Braidwood Recreation Club[(7)]	2 NE	2,350	600
South Wilmington Sportsmens Club[(8)]	3 SSE	1,750	600
Area 1 Outdoor Club	3.5 N	*	*
Wilmington Recreation Area Club[(6)]	3.5 NNE	750	3,000

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* Information not available.

TABLE 2.1-4 (Cont'd)

FACILITY	APPROXIMATE DISTANCE AND DIRECTION FROM THE SITE (mi)	TOTAL MEMBERSHIP, FAMILIES	ESTIMATED PEAK DAILY ATTENDANCE, PERSONS
Ponderosa Sportsmans Club(9)	4 S	207	15-25
South Wilmington Fireman Beach and Park Club(6)	4 SSW	1,800	2,100
Will County Sportsmens Club(10)	4 NE	550	800
Fossil Rock Recreation Club	4.5 NNE	*	*
CECo Employees Recreation Association, Inc.(11)	5 NNW	500	1,000
Coal City Area Club(6)	5 NNW	1,600	4,500
Sun Recreation Club	5 S	*	*
Shannon Shores	6 S	*	*
Dresden Lakes Sports Club (Public)(6)	7 NNW	*	350
Rainbow Council Scout Reservation(12)	7 NW	6,000	1,000
Goose Lake Club(13)	7.5 NNW	736	500

SOURCES

(1) "Land & Historic Sites Attendance," Illinois Department of Conservation, December 1976.

(2) Mr. D. Doyle, Des Plaines Conservation Area, Telephone Conversation with J. M. Ruff, Cultural Resource Analyst, Sargent & Lundy, March 21, 1977.

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[*] Information not available.

TABLE 2.1-4(Cont'd)

(3) Mr. J. Nyhoff, Goose Lake Priare State Park, Telephone Conversation with J. M. Ruff, Cultural Resource Analyst, Sargent & Lundy, March 21, 1977.

(4) Mrs. Classen, Kankakee River State Park, Telephone Conversation with J. M. Ruff, Cultural Resource Analyst, Sargent & Lundy, March 21, 1977.

(5) Mr. B. Schwiesow, Ranger-I&M Canal Complex, Telephone Conversation with J. M. Ruff, Cultural Resource Analyst, Sargent & Lundy, March 23, 1977.

(6) Preliminary Safety Analysis Report, Braidwood Station, Table 2.1-8, p. 2.1-23.

(7) Ms. B. Chilman, Braidwood Recreation Club, Telephone Conversation with J. M. Ruff, Cultural Resource Analyst, Sargent & Lundy, March 22, 1977.

(8) Mr. J. Dvorak, South Wilmington Sportsmans Club, Telephone Conversation with J. M. Ruff, Cultural Resource Analyst, Sargent & Lundy, March 23, 1977.

(9) Mr. E. Woolwine, Secretary-Treasurer Ponderosa Sportsmans Club, Letter to J. M. Ruff, Cultural Resource Analyst, Sargent & Lundy, May 1, 1977.

(10) Ms. M. Burdick, Will County Sportsmen's Club, Telephone Conversation with J. M. Ruff, Cultural Resource Analyst, Sargent & Lundy, March 21, 1977.

(11) Mr. R. Errek, President, CECo Employees Recreation Association, Inc., Telephone Conversation with M. Tenner, Env. Affairs, CECo, March 25, 1977.

(12) Mr. J. Abert, Program Director, Boy Scouts of America, Telephone Conversation with J. M. Ruff, Cultural Resource Analyst, Sargent & Lundy, March 22, 1977.

(13) Ms. K. Tagliatti, Goose Lake Club, Telephone Conversation with J. M. Ruff, Cultural Resource Analyst, Sargent & Lundy, April 27, 1977.

TABLE 2.1-5

INDUSTRIES WITHIN 10 MILES OF THE SITE

NAME OF FIRM	LOCATION	EMPLOYMENT	PRODUCTS
Bailey Printing & Publishing	Coal City*	15	Commercial and job printing
Bowers-Siemon Chemicals Co.	Coal City	30	Industrial lubricants and chemicals for wire industry
Coal City Ready Mix	Coal City	8	Ready-mix Cement
DeMert & Dougherty Inc.	Coal City	110-115	Aerosols, etc.
Brownie Special Products Co.	Gardner**	50	Pizza crusts
Lindamood Sheet Metal	Wilmington***	6	Custom sheet metal ducts and fittings
Earl A Muser & Co.	Wilmington	under 5	Tools and dies
Personal Products Co. Division of Johnson & Johnson	Wilmington	300-350	Hygienic products
Exelon Generation Company Training Center	Wilmington (RR 2, Essex Rd.)	100	Production training average enrollment 150 trainees

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Source: Commonwealth Edison Company (1982).

* Coal City is 3.5 miles northwest of the station.
** Gardner is 5.5 miles southwest of the station.
*** Wilmington is 6.0 miles northeast of the station.

TABLE 2.1-6

SCHOOLS WITHIN 10 MILES OF THE SITE

INSTITUTIONS	DISTANCE AND DIRECTION FROM SITE	GRADES	ENROLLMENT 1981-1982	STAFF 1981-1982
Braidwood, Illinois				
Braidwood Elementary and Middle School	1.4 miles NNE	K-8	712	37
Reed Custer High	1.4 miles NNE	9-12	365	23
Braceville, Illinois				
Braceville Elementary	2.0 miles SW	K-8	164	11
Coal City, Illinois				
Coal City Elementary	3.5 miles NW	K-5	742	42
Coal City High	3.5 miles NW	9-12	471	32
Coal City Middle	3.5 miles NW	6-8	369	24
Essex, Illinois				
Essex Elementary	5.0 miles SSE	1-5	75	4
South Wilmington, Illinois				
South Wilmington Consolidated Elementary	5.2 miles SSW	K-8	114	7

TABLE 2.1-6 (Cont'd)

INSTITUTIONS	DISTANCE AND DIRECTION FROM SITE	GRADES	ENROLLMENT 1981-1982	STAFF 1981-1982
<u>Gardner, Illinois</u>				
Gardner Elementary	5.3 miles SW	K-8	256	13
Gardner-South Wilmington Township High School	5.3 miles SW	9-12	264	20
<u>Custer Park, Illinois</u>				
Custer Park Elementary	5.3 miles E	K-8	172	13
<u>Wilmington, Illinois</u>				
Bruning Elementary	6.0 miles NE	K-5	287	13
L. J. Stevens Middle	6.0 miles NE	6-8	390	24
Wilmington High	6.1 miles NE	9-12	556	35
St. Rose School[a]	6.2 miles NE	1-8	222	12
Booth Central Elementary	6.3 miles NE	K-5	466	22

Source: Illinois State Board of Education (1982).

[a] Source: Florella (1982).4

TABLE 2.1-7

1980 AND PROJECTED POPULATION DISTRIBUTION BETWEEN 0-10 MILES OF THE BRAIDWOOD SITE INCLUDING PEAK DAILY TRANSIENT POPULATION

SECTOR DESIGNATION	1980	1990	2000	2010	2020
N	3,840 (1,439+2,401*)	4,213 (1,812+2,401*)	4,339 (1,938+2,401*)	4,394 (1,993+2,401*)	4,451 (2,050+2,401*)
NNE	5,876 (2,456+3,420*)	6,443 (3,023+3,420*)	6,641 (3,221+3,420*)	6,733 (3,313+3,420*)	6,829 (3,409+3,420*)
NE	7,055 (5,425+1,630*)	7,439 (5,809+1,630*)	7,657 (6,027+1,630*)	7,829 (6,199+1,630*)	8,005 (6,375+1,630*)
ENE	2,331	2,482	2,574	2,647	2,723
E	26,090 (1,090+25,000*)	26,176 (1,176+25,000*)	26,223 (1,223+25,000*)	26,258 (1,258+25,000*)	26,293 (1,293+25,000*)
ESE	321	365	385	396	406
SE	177	205	218	224	229
SSE	1,662 (662+1,000*)	1,778 (778+1,000*)	1,822 (822+1,000*)	1,845 (845+1,000*)	1,869 (869+1,000*)
S	1,852 (692+1,160*)	1,940 (780+1,160*)	1,977 (817+1,160*)	2,000 (840+1,160*)	2,024 (864+1,160*)
SSW	3,176 (1,076+2,100*)	3,201 (1,101+2,100*)	3,231 (1,131+2,100*)	3,263 (1,163+2,100*)	3,297 (1,197+2,100*)

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*Denotes transient population only.

TABLE 2.1-7 (Cont'd)

1980 AND PROJECTED POPULATION DISTRIBUTION BETWEEN 0-10 MILES OF THE BRAIDWOOD SITE INCLUDING PEAK DAILY TRANSIENT POPULATION

SECTOR DESIGNATION	1980	1990	2000	2010	2020
SW	2,904 (2,404+500*)	3,089 (2,589+500*)	3,189 (2,689+500*)	3,266 (2,766+500*)	3,344 (2,844+500*)
WSW	714	754	779	801	825
W	1,021	1,091	1,131	1,165	1,198
WNW	317	365	384	396	408
NW	4,838 (3,838+1,000*)	5,588 (4,588+1,000*)	5,865 (4,865+1,000*)	6,004 (5,004+1,000*)	6,146 (5,146+1,000*)
NNW	15,725 (3,519+12,206*)	17,057 (4,851+12,206*)	17,477 (5,271+12,206*)	17,628 (5,422+12,206*)	17,781 (5,575+12,206*)
Sum for 0-10 mile interval	77,899 (27,482+50,417*)	82,186 (31,769+50,417*)	83,892 (33,475+50,417*)	84,849 (34,432+50,417*)	85,828 (35,411+50,417*)
Average density (persons/mi^2) in 0-10-mile interval	248	262	267	270	273

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*Denotes transient population only.

TABLE 2.1-8

1980 AND PROJECTED POPULATION DISTRIBUTION WITHIN THE LPZ INCLUDING TRANSIENT POPULATION

SECTOR DESIGNATION	1980	1990	2000	2010	2020
N	68	88	94	96	100
NNE	113	146	156	161	165
NE	0	0	0	0	0
ENE	4	5	6	6	6
E	0	0	0	0	0
ESE	0	0	0	0	0
SE	0	0	0	0	0
SSE	0	0	0	0	0
S	0	0	0	0	0
SSW	0	0	0	0	0
SW	902 (402 + 500*)	978 (478 + 500*)	1,006 (506 + 500*)	1,020 (520 + 500*)	1,035 (535 + 500*)

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*Denotes transient population only. (See NOTE)

TABLE 2.1-8 (Cont'd)

1980 AND PROJECTED POPULATION DISTRIBUTION WITHIN THE LPZ INCLUDING TRANSIENT POPULATION

SECTOR DESIGNATION	1980	1990	2000	2010	2020
WSW	98	112	119	123	126
W	0	0	0	0	0
WNW	8	8	8	9	9
NW	4	5	6	6	6
NNW	12	16	16	18	18
Sum for LPZ	1,205 (705 + 500*)	1,358 (858 + 500*)	1,411 (911 + 500*)	1,439 (939 + 500*)	1,465 (965 + 500*)
Average density (persons/mi^2) in LPZ	303	342	355	362	368

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*Denotes transient population only. (See NOTE)

NOTE:

P. Zidich, Co-President Chicago Beagle Club, Telephone Conversation with B. Barickman, Commonwealth Edison, October 28, 1982.

TABLE 2.1-9

POPULATION CENTERS WITHIN 50 MILES OF THE SITE
(1980)

POPULATION[*] CENTER	COUNTY	1980 POPULATION	2020 POPULATION
Joliet	Will	77,956	82,501
Kankakee	Kankakee	30,141	31,065
Park Forest	Will and Cook	26,222	35,023
Aurora	Kane	81,293	92,830
Chicago Heights	Cook	37,026	43,046
Downers Grove	DuPage	39,274	53,334
Harvey	Cook	35,810	39,869
Oak Lawn	Cook	60,590	66,883
Wheaton	DuPage	43,043	57,640
Calumet City	Cook	39,673	43,908
Chicago (part)	Cook	3,005,072	2,847,231
Lombard	DuPage	37,295	41,175
Hammond	Lake (Ind.)	93,714	98,434
Elmhurst	DuPage	44,251	50,140
Maywood	Cook	27,998	27,136
Tinley Park	Will and Cook	26,171	39,936
Highland	Lake (Ind.)	25,935	27,241
East Chicago	Lake (Ind.)	39,786	41,790
Oak Forest	Cook	26,096	32,529
Lansing	Cook	29,039	32,444
Addison	DuPage	28,836	37,059
Bolingbrook	DuPage and Will	37,261	64,928
Naperville	DuPage and Will	42,330	65,976
Berwyn	Cook	46,849	44,418
Cicero	Cook	61,232	59,853

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[*] A population center is defined as an urban area having 25,000 or more persons.

TABLE 2.1-10

URBAN CENTERS WITHIN 30 MILES OF THE SITE
(1980)

URBAN CENTER[*]	COUNTY	DISTANCE AND DIRECTION FROM SITE	1980 POPULATION	2020 POPULATION
Coal City	Grundy	3.5 miles NW	3,028	3,898
Wilmington	Will	6.0 miles NE	4,424	5,032
Morris	Grundy	13 miles NW	8,833	9,954
Dwight	Livingston	14 miles SW	4,146	4,905
Bourbonnais	Kankakee	19 miles ESE	13,280	18,776
Bradley	Kankakee	20 miles ESE	11,008	15,564
Joliet	Will	20 miles NNE	77,956	82,501
Kankakee	Kankakee	20 miles ESE	30,141	31,065
Manteno	Kankakee	20 miles E	3,155	1,077
Crest Hill	Will	22 miles NNE	9,252	10,907
New Lenox	Will	23 miles NE	5,792	8,916
Lockport	Will	26 miles NNE	9,017	10,188
Plainfield	Will	25 miles N	4,485	6,160
Romeoville	Will	28 miles NNE	15,519	23,172
Momence	Kankakee	30 miles E	3,297	4,001
Marseilles	La Salle	25 miles WNW	4,766	5,659
Channahon	Will	13 miles N	3,734	5,791
Frankfort	Will	26 miles NE	4,357	7,374
Mokena	Will	26 miles NE	4,578	7,748
Peotone	Will	23 miles ENE	2,832	3,507
Shorewood	Will	20 miles N	4,714	7,556
Braidwood	Will	1.3 miles N	3,429	5,026

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[*] An urban center is defined as an incorporated or an unincorporated place with a population of over 2500 according to the 1980 census.

TABLE 2.1-11

AVERAGE NUMBER OF PEOPLE PER HOUSEHOLD IN TOWNSHIPS WITHIN 10 MILES OF SITE

COUNTIES (TOWNSHIPS)	AVERAGE NO. OF PEOPLE PER HOUSEHOLD*
Will County	
Channahon	2.7
Custer	3.1
Florence	3.6
Reed	2.0
Wesley	3.5
Wilmington	2.0
Grundy County	
Braceville	2.8
Felix	3.2
Garfield	3.4
Goodfarm	2.9
Goose Lake	3.8
Greenfield	2.6
Maine	3.3
Mazon	2.7
Wauponsee	3.1
Kankakee County	
Essex	2.7
Norton	2.9
Salina	3.1

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*Numbers based on U.S. Census Bureau's 1980 population statistics and the number of housing units in the unincorporated areas of each township.

2.2 NEARBY INDUSTRIAL, TRANSPORTATION, AND MILITARY FACILITIES

2.2.1 Locations and Routes

The major transportation routes within 5 miles of the Braidwood Station include highways and railroads, as shown in Figure 2.1-6. The Kankakee River located approximately 3 miles east of the northeastern site boundary is primarily used for recreational purposes (Reference 1).

The nearest highways to the site, Illinois State Routes 53 and 129, are adjacent to the northwest boundary of the site. Interstate 55 is less than 2 miles west-northwest of the site (centerline of the reactors), and State Route 113 is approximately 2 miles north of the site. Figure 2.1-6 illustrates these highways and their traffic volumes. Relatively high traffic flow occurs on Interstate 55, with a 24-hour annual average of 13,700 cars. State Routes 129 and 113 are also well traveled, having 24-hour annual averages of 2,900 and 4,300, respectively near their interchanges with Interstate 55. State Route 53 has 24-hour annual averages varying from 600 to 4,600 cars within 5 miles of the site.

As shown in Figure 2.1-6, there are four railroads within 5 miles of the site. The Illinois Central Gulf Railroad (ICG) runs parallel to and between State Routes 53 and 129 and provides spur track access to the site. This line, which is a secondary freight route, has six passenger trains per day (three northbound and three southbound), six piggy-backs per day (three northbound and three southbound) and an occasional northbound coal train, in addition to shipments to and from the Joliet Army Ammunition Plant, as discussed in Subsection 2.2.2.2 (Reference 2). The Illinois Central Gulf Railroad also has a line located approximately 2.5 miles west of the site. Both tracks are designated as Class 4 by the ICG in accordance with Federal Railroad Administration (FRA) track safety standards (Reference 3).

In addition to the ICG, rail transportation within 5 miles of the site is provided by the Norfolk and Western Railroad (N&W), located approximately 4.5 miles southeast of the site, and the Atchison, Topeka and Santa Fe Railroad (AT&SF), located approximately 4 miles northwest of the site.

The N&W does not operate any scheduled passenger trains over the portion of the railroad near the site. The maximum allowable speed for freight trains over this segment of trackage is 60 mph or FRA Class 4 track (Reference 4).

All industries within 10 miles of the site are listed in Table 2.1-5. Airports and low-altitude federal airways within 10 miles of the site are listed in Tables 2.2-1 and 2.2-2 respectively and are located on Figure 2.2-1. Pipelines within 5 miles of the site are listed in Table 2.2-3 and illustrated on Figure 2.2-2.

The Braidwood Station is located approximately 8 miles southwest of the Joliet Army Ammunition Plant. There are no military bases, missile sites, or military firing or bombing ranges within 10 miles of the site.

2.2.2 Descriptions

2.2.2.1 Description of Facilities

All industries within 10 miles of the site are listed in Table 2.1-5 along with their respective products and approximate number of employees. Table 2.2-4 lists users of hazardous materials within 5 miles of the site. There are no manufacturers of hazardous materials identified within 5 miles of the site.

As shown in Table 2.1-5, the area within 10 miles of the site is not heavily industrialized. The nearest industries are located in Coal City, Illinois, approximately 3.5 miles northwest of the site.

2.2.2.2 Description of Products and Materials

Table 2.1-5 lists all industries located within 10 miles of the site. The industries within 5 miles of the site that deal with hazardous materials are listed in Table 2.2-4 with the relevant hazardous materials used or stored, as well as their maximum quantities and modes of transportation.

The Joliet Army Ammunition Plant, located approximately 8 miles northeast of the Braidwood Station, produces medium caliber ammunition. Ammunition and propellant are shipped by truck and these trucks are not routed on Illinois State Highway 53 or 129 past the Braidwood plant (Reference 15). Reference 16 determined the Union Pacific Railroad does not ship any explosives on the rail line past the Braidwood site.

The Joliet Army Ammunition Plant is currently being used for storage of explosive material. There are 611,000 ft^2 of earth-covered magazines available for storage of "raw material (hazardous) and 351,000 ft^2 of aboveground magazines available for storage of end items." As of March 31, 1977, 42% of the raw material storage capability was being utilized, and 86% of the end item storage capability was being utilized (Reference 6).

2.2.2.3 Pipelines

There are six natural gas pipelines, three crude oil pipelines, and one refined products pipeline within 5 miles of the site. Figure 2.2-2 locates these pipelines, and Table 2.2-3 summarizes the pipe size, pipe age, operating pressure, depth of burial, and location and type of isolation valves for each buried pipeline. These pipelines are used for transport and are unlikely to be used to store or transport any other material than that currently being transported.

There are no tank farms within 5 miles of the site.

2.2.2.4 Waterways

As shown in Figure 2.1-6, the Kankakee River at its closest point is approximately 3 miles east of the northeastern site boundary. This point is approximately 12 miles upstream from the headwaters of the Illinois River (confluence of the Kankakee and Des Plaines Rivers).

The Kankakee River is considered to be Navigable Waters of the United States ("has been, is, or could be used as a highway in interstate commerce") for the first 5-1/2 miles upstream from the headwaters of the Des Plaines River. The remainder of the river, including the portion near the Braidwood Station, is considered navigable waters and thus falls under the U.S. Corps of Engineers permit procedures. There are no barge statistics available for the Kankakee River, since no commercial barges have been on the portion which is considered Navigable Waters of the United States for many years. There is no designated shipping channel for the portion of the river which is Navigable Waters of the United States, and the rest of the river is considered navigable from bank to bank. There is no designated depth of channel for any portion of the river. The nearest dams to the site are fixed dams located at Wilmington (downstream from the site) and Kankakee (upstream from the site) (Reference 7). The U.S. Corps of Engineers maintains no dams or locks on the Kankakee River (Reference 1). The Kankakee River is used primarily for recreational purposes (Reference 1).

2.2.2.5 Airports

There are no airports within 5 miles of the site. There are three private airports within 10 miles of the site, as listed in Table 2.2-1. As indicated in the table, these airports have turf runways with few operations daily. Figure 2.2-1 locates the airports within 10 miles. The nearest airport having a paved runway is the Dwight Airport, located approximately 13 miles southwest of the site, near Dwight, Illinois. Commercial service for the region is provided by O'Hare International, Chicago Midway and Merrill C. Meigs Field airports.

There are no airports within 10 miles of the site with projected operations greater than $500d^2$ (d = distance in miles) movements per year, nor are there any airports with projected operations greater than $1000d^2$ per year outside 10 miles.

There are two Low-Altitude Federal Airways within 10 miles of the site. Figure 2.2-1 locates these airways with respect to the site. These airways are 8 nautical miles wide and are between radio stations. There are two types of minimum altitude requirements for Low-Altitude Federal Airways, minimum obstruction clearance altitude (terrain clearance) and minimum en-route altitude (radio reception). Table 2.2-2 summarizes the minimum altitude requirements for the two airways near the site. The Low-Altitude Federal Airways have a maximum altitude of 18,000 feet (Reference 8).

2.2.2.6 Projections of Industrial Growth

All industries within 10 miles of the site are listed in Table 2.1-5. There are no known plans for expansion of these industries in the immediate future.

All airports within 10 miles of the site are listed in Table 2.2-1. At the present time, there are no known plans for expansion for these airports.

Table 2.2-3 lists the pipelines within 5 miles of the site. There is no planned expansion for any pipelines near the site.

2.2.3 Evaluation of Potential Accidents

On the basis of the information provided in Subsections 2.2.1 and 2.2.2, safety evaluations of the activities described therein are provided in the following subsections.

2.2.3.1 Determination of Design-Basis Events

The accident categories discussed below have been evaluated.

2.2.3.1.1 Explosions

Potential hazards involving the detonation of high explosives, munitions, chemicals, or liquid and gaseous fuels for facilities and activities in the vicinity of the plant where such materials are processed, stored, used, or transported in quantity have been evaluated. In Reference 14, a probabilistic safety analysis per the requirements of Regulatory Guide 1.91 was performed to evaluate the potential for an explosion of TNT on transportation routes near the Braidwood site. The evaluation considered truck traffic on Illinois State Highways 53 and 129 and railroad traffic on the Union Pacific Railroad line that is located between Illinois State Highways 53 and 129. Other transportation routes are sufficiently distant from the Braidwood site to preclude significant TNT blast effects at the station location.

Conservatively assuming that 12 trains per year on the Union Pacific Railroad line carry significant quantities of TNT, Reference 14 determined the probability of a railroad-related TNT explosion near the Braidwood site is 2.0×10^{-8}. This is significantly less than the acceptance criterion per Regulatory Guide 1.91. Therefore, a TNT explosion on the rail line near the Braidwood site is not a credible event.

The probability of a TNT explosion on Illinois State Highway 53 or 129 adjacent to the plant site is 2.6 x 10^{-7}. This evaluation was based on explosive material incident information obtained from the United States Department of Transportation, total truck miles driven in the United States per year, truck traffic volume on Illinois State Highways 53 and 129 near the Braidwood site, and the length of these highways where a truck quantity (50,000 pound) TNT explosion could have an adverse effect on the plant. Based on this evaluation, a TNT explosion related to highway traffic near the Braidwood site is not a credible event.

The overall probability of a TNT explosion on transportation routes near the Braidwood site, based on conservative evaluation methods, is 2.8 x 10^{-7}, which is less than the 1.0 x 10^{-6} acceptance criterion of Regulatory Guide 1.91. Therefore, an accidental explosion of TNT on transportation routes near the Braidwood site does not need to be considered as a design-basis event.

2.2.3.1.2 Flammable Vapor Clouds (Delayed Ignition)

There is no possibility of an accident that could lead to the formation of flammable vapor clouds in the vicinity of the plant because (1) there is no industry in the vicinity of the plant which can produce a flammable vapor cloud, (2) there is no pipeline of sufficient size in the vicinity of the plant which can produce a flammable vapor cloud, and (3) there are no tank farms in the vicinity of the plant.

2.2.3.1.3 Toxic Chemicals

Various chemicals in varying amounts are stored on site. The largest amounts of these chemicals are required for treatment of raw water systems, such as circulating water, essential and nonessential service water. The storage of these chemicals, the effects on plant systems and operations have been evaluated.

Chemical Hazards stored and transported in the vicinity of the plant are analyzed in accordance with Technical Specification 5.5.18 and Regulatory Guides 1.78, 1.95 and 1.196 to ensure Control Room Habitability is maintained. The latest analysis was performed in 2008.

The analysis concluded that toxic chemicals transported or stored within the vicinity of the plant do not pose a threat that requires use of instrumentation. The potential chemical hazards were dispositioned based upon dispersion calculations verified toxicity limits were not exceeded, the frequency of shipments met Regulatory Guide 1.78, and more than 2 minutes elapsing between the time of nasal detection to the time when the IDLH (Immediately Dangerous to Life and Health) limit is reached. There were 4 chemicals that resulted in relying on nasal detection (Propane, Anhydrous Ammonia, Chlorine, and Gasoline).

Potential chemical hazards identified are listed in Table 2.2-4 for used or stored hazardous material within 5 miles of the site and in Table 2.2-5 for shipments of toxic chemicals within 5 miles of the site.

2.2.3.1.4 Fires

No fire hazard threatens the plant safety since no chemical plants, no large amounts of oil storage, and no gas pipelines are located in the vicinity of the plant. The potential for deleterious effects from forest or brush fires is minimized by the site's landscaping.

Onsite fire hazards are discussed in Subsection 9.5.1.

2.2.3.1.5 Collisions with Intake Structure

There is no potential for a barge or ship impact on the river makeup screen house, since the Kankakee River is nonnavigable in the vicinity of the site.

2.2.3.1.6 Liquid Spills

No potential for the accidental release of oil or liquids which may be corrosive, cryogenic, or coagulant, and which may be drawn into the plant's intake structure and circulating water system or which may otherwise affect the safety of the plant has been found.

The drainage system in the vicinity of the chemical feed system on the west side of the Lake Screen House is designed to route any chemical spillage away from the intake structure and to the circulating water discharge portion of the cooling pond.

2.2.4 References

1. Russell Carlock, Joliet Project Office, Corps of Engineers, Department of the Army, Telephone Conversation with J. M. Ruff, Cultural Resource Analyst, Sargent & Lundy, April 6, 1977.

2. John L. Turnland, Superintendent Yards and Terminals, Illinois Central Gulf Railroad, Telephone Conversation with J. M. Ruff, Cultural Resource Analyst, Sargent & Lundy, June 28, 1977.

3. John L. Turnland, Superintendent Yards and Terminals, Illinois Central Gulf Railroad, Letter to J. M. Ruff, Cultural Resource Analyst, Sargent & Lundy, June 7, 1977.

4. John P. Fishwick, President and Chief Executive Officer, Norfolk and Western Railroad, Letter to J. M. Ruff, Cultural Resource Analyst, Sargent & Lundy, June 20, 1977.

5. Deleted.

6. Robert J. Surkein, Director, Transportation and Traffic Management Directorate, Department of the Army, Headquarters United States Army Armament Material Readiness Command, Rock Island, Illinois, Letter to J. M. Ruff, Cultural Resource Analyst, Sargent & Lundy, June 20, 1977.

7. Betty Klemba, Corps of Engineers, Department of the Army, Telephone Conversation with J. M. Ruff, Cultural Resource Analyst, Sargent & Lundy, April 6, 1977.

8. Bonnie Ferguson, Operations Specialist, General Aviation District Office, Federal Aviation Administration, Telephone Conversation with J. M. Ruff, Cultural Resource Analyst, Sargent & Lundy, April 19, 1977.

9. Braidwood-PSAR, Appendix A to Chapter 2.0.

10. R. J. Surkein, Director, Transportation and Traffic Management Directorate, Department of the Army Material Readiness, Rock Island, Illinois, Letter to J. M. Ruff, Cultural Resource Analyst, Sargent & Lundy, June 20, 1977.

11. Braidwood-PSAR, Appendix B to Chapter 2.0.

12. Deleted.

13. Deleted.

14. Evaluation of the probability of TNT explosion at Braidwood, Calculation BRW-98-0173-M, Revision 0

15. Letter from Jeffrey L. Smetters, Commonwealth Edison to David Geiss, Alliant TechSystems, Inc. at the Joliet Arsenal, Letter SG-98-0008-BRW, January 20, 1998.

16. Letter from Sandra S. Covi, Manager of Hazardous Materials Management, Union Pacific Railroad to Jeffrey Smetters, Commonwealth Edison dated March 3, 1998.

TABLE 2.2-1

AIRPORTS WITHIN 10 MILES OF THE SITE

AIRPORT	APPROXIMATE DISTANCE AND DIRECTION FROM THE SITE	NUMBER OF BASED AIRCRAFT	HOURS	APPROXIMATE OPERATIONS	RUNWAYS	TYPE	LENGTH (ft)	WIDTH (ft)
Matteson RLA (Private)	6 miles W	2 single engine	Unattended	no more than 2 daily*	18/36	Turf	2200	100
J. B. Fillman (Private)	7 miles WSW	2 single engine	Intermittent	no more than 2 daily*	N/S	Turf	1800	125
Hugh Van Voorst (Private)	10 miles SSE	1 Multi-engine	Unattended	no more than 2 daily*	09/27	Turf	3450	120

Source: FAA Form 5010-1 for each airport (Matteson RLA, November 9, 1976, J. B. Fillman December 8, 1976, Hugh Van Voorst, December 8, 1976).

* Letter from Michael C. Rose, Airports Planning Specialist, Chicago Airports District Office, Federal Aviation Administration, to C. Comerford, Cultural Resource Analyst, Sargent & Lundy, April 15, 1977.

TABLE 2.2-2

LOW ALTITUDE FEDERAL AIRWAYS WITHIN 10 MILES OF THE SITE

AIRWAYS	MINIMUM OBSTRUCTION CLEARANCE ALTITUDE (TERRAIN)	MINIMUM EN-ROUTE ALTITUDE (RADIO)
V156 Peotone - Bradford	2100 ft.	2600 ft.
V429 Roberts - Joliet	2100 ft.	2500 ft.

Source: B. Ferguson, Operations Specialist, FAA-General Aviation District Office, Telephone Conversation with J. M. Ruff, Cultural Resource Analyst, Sargent & Lundy, April 19, 1977.

TABLE 2.2-3

PIPELINES WITHIN 5 MILES OF THE SITE

PIPELINE COMPANY	PIPE SIZE (in)	MATERIAL CARRIED	APPROXIMATE PIPE AGE (yr)	OPERATING PRESSURE (psi)	APPROXIMATE DEPTH OF BURIAL (ft)	LOCATION AND TYPE OF ISOLATION VALVES
Arco Pipeline Company(1)	8	Refined* Products	25-74	450	3	Manual block valves location depends upon terrain
Midwestern Gas Transmission Line Company(2)	30	Natural gas	18	700-800	2.5 or more	**
Natural Gas Pipeline Company of America(3)	36	Natural gas	24	Designed for 858 maximum. Normally does not operate at maximum	3.5	Automatic valves located every 10 miles
Northern Illinois Gas Company(4)	4	Natural gas	13	60	3	Manual valve located at least every 10 miles
	6	Natural gas	6-9	Designed for 230 operating at 150	3	Manual valve located at least every 10 miles
	12	Natural gas	**	60	3	Manual valve located at least every 10 miles
	36(5)	Natural gas	12	750	3	Manual valve located at least every 10 miles

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* Refined products - gasoline, kerosene, LPG, and ammonia

** Information not available.

TABLE 2.2-3 (Cont'd)

PIPELINES WITHIN 5 MILES OF THE SITE

PIPELINE COMPANY	PIPE SIZE (in)	MATERIAL CARRIED	APPROXIMATE PIPE AGE (yr)	OPERATING PRESSURE (psi)	APPROXIMATE DEPTH OF BURIAL (ft)	LOCATION AND TYPE OF ISOLATION VALVE
Texaco-Cities Service Pipeline Company(6)	12	Crude Oil	48	720	2-3	Manual valves located at pump stations and major streams
	12	Crude Oil	40	750	2-3	Manual valves located at pump stations and major streams
	18	Crude Oil	28	850	2-3	Manual valves located at pump stations and major streams

SOURCES

1. A. F. Morel, Arco Pipeline Company, Mazon District Office, Telephone Conversation with J. M. Ruff, Cultural Resource Analyst, Sargent & Lundy, April 4, 1977.
2. L. Howard, Midwestern Gas Transmission Line Company, Telephone Conversation with J. M. Ruff, Cultural Resource Analyst, Sargent & Lundy, April 6, 1977.
3. M. Harbach, Natural Gas Pipeline Company of America, Telephone Conversation with J. M. Ruff, Cultural Resource Analyst, Sargent & Lundy, April 4, 1977.
4. Mr. R. Mores, Northern Illinois Gas Company, Telephone Conversation with J. M. Ruff, Cultural Resource Analyst, Sargent & Lundy, April 1, 1977.
5. Mr. B. Weirich, Northern Illinois Gas Company, Telephone Conversation with J. M. Ruff, Cultural Resource Analyst, Sargent & Lundy, April 4, 1977.
6. Mr. H. M. Miller, Texaco-Cities Service Pipeline Company, Division Manager, Letter to J. M. Ruff, Cultural Resource Analyst, Sargent & Lundy, May 18, 1977.

TABLE 2.2-4

INDUSTRIES WITH HAZARDOUS MATERIALS WITHIN 5 MILES OF THE SITE

INDUSTRY	LOCATION	APPROXIMATE DISTANCE AND DIRECTION FROM THE SITE	MATERIAL	MAXIMUM QUANTITY STORED (pounds)	TOXICITY LIMITS* (mg/m3)	MODE OF TRANSPORTATION (lbs/shipments per year)
Chicago Aerosol	Coal City	4 miles NW	Acetone	84,000	4800	truck (40,000/13)
Chicago Aerosol	Coal City	4 miles NW	Dimethyl Ether	72,000	Not Established	truck (40,000/4)
Chicago Aerosol	Coal City	4 miles NW	Butane	120,000	2400	truck (40,000/36)
Chicago Aerosol	Coal City	4 miles NW	Isobutane	120,000	Asphyxiant	truck (40,000/58)
Chicago Aerosol	Coal City	4 miles NW	1, 1Difluoroethane	205,000	Asphyxiant	truck (40,000/83)
Chicago Aerosol	Coal City	4 miles NW	Propane	118,000	3780	truck (40,000/25)
Hicks Gas	Braidwood	0.9 miles NE	Propane	161,032	3780	See Table 2.2-5
Nicor Gas	Wilmington	3.8 miles E	Natural Gas	13,340	N/A	gas line

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Source: Braidwood Calculation BRW-08-0075-M

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TABLE 2.2-5

SHIPMENTS OF TOXIC CHEMICALS WITHIN 5 MILES OF THE SITE

TOXIC CHEMICAL	QUANTITY (LBS per shipment)	APPROXIMATE DISTANCE FROM THE SITE (miles)	MODE OF TRANSPORTATION	SHIPMENTS (per year)
Acetone	184,530	3.7	Rail	25
Anhydrous Ammonia	173,382	3.7	Rail	547
Butane	165,635	3.7	Rail	326
Carbon Dioxide	155,045	3.7	Rail	66
Chlorine	234,729	3.7	Rail	233
Ethyl Ether	59,532	3.7	Rail	1
Ethylene Oxide	74,246	3.7	Rail	64
Methonal	197,968	3.7	Rail	337
Sulfuric Acid	213,930	3.7	Rail	522
Xylene	193,707	3.7	Rail	23
Chlorine	2,000	1.1	Highway	(Note 1)
Large Propane Truck	39,312	0.24	Road	(Note 1)
Small Propane Truck	11,050	0.24	Road	(Note 1)
Gasoline	44,500	0.24	Road	(Note 1)
Farmer Ammonia	1,450	0.24	Road	(Note 1)

Note 1: Not specified. Evaluated acceptable based upon nasal detection.

Source: Braidwood Calculation BRW-08-0075-M

2.3 METEOROLOGY

Section 2.3 provides a meteorological description of the Braidwood Station site and its surrounding areas. Included are a description of the general climate, meteorological conditions used for design and operating-basis consideration, summaries of normal and extreme values of meteorological parameters, a discussion of the potential influence of the plant and its facilities on local meteorology, a description of the onsite meteorological measurements program, and short-term and long-term diffusion estimates. Detailed summaries of meteorological parameters are presented using data from Argonne National Laboratory (1950-1964), from the first-order National Weather Service Stations at Peoria, Illinois (1943-1976) and Chicago Midway Airport (1943-1976), and from the meteorological towers at the Braidwood Station site (1974-1976) and the Dresden Nuclear Power Station (1974-1976).

Based on the information presented in this section, it is concluded that there are no unusual local conditions that should adversely affect the plant operation, the dispersion of the plant effluents, and the dissipation of the plant waste heat.

2.3.1 Regional Climatology

2.3.1.1 General Climate

The Braidwood Station site is located in northeastern Illinois, approximately 86 miles east-northeast of the first-order National Weather Service Station at Peoria, Illinois, and 56 miles south-southwest of the first-order National Weather Service Station at Chicago Midway Airport. General climatological data for the region were obtained from the United States Environmental Science Service Administration (ESSA) Climate of Illinois report (Reference 1) and from the Local Climatological Data Annual Summaries for the first-order weather stations at Peoria and Chicago Midway (References 2 and 3). The 15-year climatological summary for Argonne National Laboratory, which is located approximately 34 miles north-northeast of the Braidwood site, was also consulted for specific statistics (Reference 4).

Although Chicago Midway Airport is located somewhat closer to the Braidwood site than Greater Peoria Airport, the latter is considered to be more representative of the climate at the Braidwood site. This is because the moderating influence of Lake Michigan is considerable at Chicago Midway, while at the more inland sites it is much less. The climate of northeastern Illinois is typically continental, with cold winters, warm summers, and frequent short-period fluctuations in temperature, humidity, cloudiness, and wind direction. The great variability in northern Illinois climate is due to its location in a confluence zone, particularly during the cooler months, between different air masses (Reference 5). The specific air masses which affect northeastern Illinois include maritime tropical air which originates in the Gulf of Mexico, continental tropical air which originates in Mexico and the southern Rockies, Pacific air which originates in the eastern North Pacific Ocean, and continental polar and continental arctic air which originate in Canada. As these air masses migrate from their source regions, they may undergo substantial modification in their characteristics. Monthly streamline analyses of resultant surface winds suggest that air reaching northeastern Illinois most frequently originates over the Gulf of Mexico from April through August, over the southeastern United States from September through November, and over both the Pacific Ocean and the Gulf of Mexico from December through March (Reference 5).

The major factors controlling the frequency and variation of weather types in northeastern Illinois are distinctly different during two separate periods of the year.

During the fall, winter, and spring months, the frequency and variation of weather types are determined by the movement of synoptic-scale storm systems which commonly follow paths along a major confluence zone between air masses, which is usually oriented from southwest to northeast through the region. The confluence zone normally shifts in latitude during this period, ranging in position from the central states to the U.S.-Canadian border. The average frequency of passage of storm systems along this zone is about once every 4 to 8 days. The storm systems are most frequent during winter and spring months, causing a maximum of cloudiness during these seasons. Winter is characterized by alternating periods of steady precipitation (rain, freezing rain, sleet and snow) and periods of clear, crisp, and cold weather. Springtime precipitation is primarily showery in nature. The frequency passage of storm systems, presence of high winds aloft, and frequent occurrence of unstable conditions caused by the close proximity of warm, moist air masses to cold, dry air masses result in this season's relatively high frequency of thunderstorms. These thunderstorms on occasion are the source for hail, damaging winds and tornadoes. Although synoptic-scale storm systems also occur during the fall months, their frequency of occurrence is less than in winter or spring. Periods of pleasant dry weather characterize the fall season, which ends rather abruptly with the returning storminess which usually begins in November.

In contrast, weather during the summer months is characterized by weaker storm systems which tend to pass to the north of Illinois. A major confluence zone is not present in the region, and the region's weather is characterized by much sunshine with thunderstorm situations. Showers and thunderstorms are usually of the air-mass type, although occasional outbreaks of cold air bring precipitation and weather typical of that associated with the fronts and storm systems of the spring months.

When southeast and easterly winds are present in northeastern Illinois, they usually bring mild and wet weather. Southerly winds are warm and showery, westerly winds are dry with moderate temperatures, and winds from the northwest and north are cool and dry.

The prevailing wind is southerly at Peoria and westerly at Midway. Although these are the most frequent directions, the frequency of winds from other directions is relatively well distributed. The monthly average wind speed is lowest during late summer at both stations, with the prevailing direction from the south at Peoria and the southwest at Midway. The monthly average wind speed is highest during late winter and early spring at both stations, with the prevailing direction from the west-northwest and the south at Peoria and the west at Midway.

Table 2.3-1 presents a summary of climatological data from meteorological stations surrounding the Braidwood Station site. The annual average temperature at Peoria is 50.8°F, while extreme temperatures range from a maximum of 102°F to a minimum of -20°F. Maximum temperatures equal or exceed 90°F nearly 20 times per year, while minimum temperatures are less than or equal to 32°F about 130 times per year.

Humidity varies with wind direction, being lowest with west or northwest winds and higher with east or south winds. At Peoria the early morning relative humidity is highest during the late summer, with an average of 87%. The relative humidity is highest throughout the day during December, ranging from 83% in early morning to 72% at noon. Heavy fog with visibility less than 0.25 mile is rare, having an average occurrence of 21 times a year. It occurs most frequently during the winter months (Reference 2).

Annual precipitation in the Braidwood site area averages 34 to 35 inches per year. For the 40-year period (1937-1976), annual precipitation has ranged from 23.99 inches in 1940 to 50.22 inches in 1973 (Reference 2). On the average, 31% of the annual precipitation occurs in the summer months of June through August, and 64% occurs in the 6 months from April through September. However, no month averages less than 4% of the annual total. Monthly precipitation totals have ranged from 13.09 inches to 0.03 inches. The maximum 24-hour precipitation recorded at Peoria was 5.52 inches in May 1927. Snowfall commonly occurs from November through March, with an average of 23.4 inches of snow annually. The monthly maximum and 24-hour maximum snowfall recorded were 18.9 inches and 10.2 inches, respectively. Points in northeastern Illinois average about 6 days of sleet per year, with an average of 2 hours of sleet on a sleet day (Reference 6).

Because of the prevailing westerly winds, the influence of Lake Michigan on the weather of northern Illinois is not significant, except for the region in the immediate vicinity of the lake shoreline. Northeasterly flow during late fall and winter may cause increased cloudiness in the Braidwood site area. The cooling effect of the lake upon the region near the Braidwood site with northeasterly flow in spring and summer is expected to be slight, since warming of the air due to solar insolation occurs rapidly as the air moves inland (Reference 1).

The terrain in northeastern Illinois is relatively flat, and differences in elevation have no significant influence on the general climate. However, the low hills and river valleys that do exist exert a small effect upon nocturnal wind drainage patterns and fog frequency.

2.3.1.2 Regional Meteorological Conditions for Design and Operating Bases

2.3.1.2.1 Thunderstorms, Hail, and Lightning

Thunderstorms occur on an average of 49 days per year at Peoria (1944-1976) and 40 days per year at Chicago Midway Airport (1943-1976) (References 2 and 3). They occur most frequently during the months of June and July, 9 and 8 days per month at Peoria, 7 and 6 days per month at Chicago Midway for June and July, respectively. Peoria averages 5 or more thunderstorm days per month throughout the season from April through September, while Chicago Midway averages 5 or more thunderstorm days per month from April through August. Both stations average 1 or fewer thunderstorm day per month from November through February. A thunderstorm day is recorded only if thunder is heard. The observation is independent of whether or not rain and/or lightning are observed concurrent with the thunder (Reference 7).

A severe thunderstorm is defined by the National Severe Storms Forecast Center (NSSFC) of the National Weather Service as a thunderstorm that possesses one or more of the following characteristics (Reference 8):

a. winds of 50 knots or more,

b. hail 3/4 inch or more in diameter, and

c. cumulonimbus cloud favorable to tornado formation.

Although the National Weather Service does not publish records of severe thunderstorms, the above referenced report of the NSSFC gives values for the total number of hail reports 3/4 inch or greater, winds of 50 knots or greater, and the number of tornadoes for the period 1955-1967 by 1° squares (latitude x longitude). The report shows that during this 13-year period the 1° square containing the Braidwood Station site had 9 hailstorms producing hail 3/4 inch in diameter or greater, 34 occurrences of winds of 50 knots or greater, and 43 tornadoes.

At least 1 day of hail is observed per year over approximately 90% of Illinois, with the average number of hail days at a point varying from 1 to 4 (Reference 9). Considerable year-to-year variation in the number of hail days is seen to occur; annual extremes at a point vary from no hail in certain years to as many as 14 hail days in other years. About 80% of the hail days occur from March through August, with spring (March through May) being the primary period of occurrence. In northern Illinois, 53% of all hail days occur in the spring (Reference 9). Total hailstorm life at a point averages about 7 minutes, with maximum storm life reported as not over 20 minutes for Illinois (Reference 6).

The frequency of lightning flashes per thunderstorm day over a specific area can be estimated by using a formula given by J. L. Marshall (Reference 10), taking into account the distance of the location from the equator:

$$N = (0.1 + 0.35 \sin \phi) \times (0.40 \pm 0.20)$$

where:

N = number of flashes to earth per thunderstorm day per km^2, and

ϕ = geographical latitude.

For the Braidwood Station site, which is located at approximately 41° north latitude, the frequency of lightning flashes (N) ranges from 0.07 to 0.20 flashes per thunderstorm day per km^2. The value 0.20 is used as the most conservative estimate of lightning frequency in the calculations that follow.

Taking the representative average number of thunderstorm days in the site region as 49 (at Peoria), the frequency of lightning flashes per km^2 per year is 9.8 as calculated below:

$$\frac{0.2 \text{ flashes}}{\text{thunderstorm day. km}^2} \times \frac{49 \text{ thunderstorm days}}{\text{year}} = \frac{9.8 \text{ flashes}}{\text{km}^2\text{. year}}$$

The area of the Braidwood Station site is 4320 acres, or about 17.5 km^2. Hence the expected frequency of lightning flashes at the site per year is 171 as calculated below:

$$\frac{9.8 \text{ flashes}}{\text{km}^2 \text{ year}} \times 17.5 \text{ km}^2 = \frac{171 \text{ flashes}}{\text{year}}$$

For the probability of a lightning strike to safety-related structures, Marshall gives the total attractive area (in $meters^2$) for a structure of length L, width W, and height H as:

$$LW + 4H (L + W) + 4H^2\pi$$

The attractive area for a structure depends on the magnitude of the lightning current and its frequency of occurrence. The formula for the total attractive area as given here assumes a lightning strike current intensity of 2×10^4 amperes with a 50% frequency of occurrence.

For the Braidwood Station, the smallest rectangle enclosing the reactor containment buildings is approximately 132.3 meters in length and 45.7 meters in width (see Braidwood - Drawings M-5 and M-14). The height of the containment building is approximately 60.7 meters. It has been assumed that the height of the entire rectangle is 60.7 meters. This issues a realistic estimate of a lightning strike on the containment structures. The attractive area of the rectangle surrounding the containment buildings is therefore approximately 0.095 km^2.

The reactor containment buildings of Braidwood Station have a probability of being struck which is equivalent to:

$$\frac{9.8\ \text{flashes}}{\text{km}^2\ \text{year}} \times 0.095\ \text{km}^2 = 0.931\ \frac{\text{flashes}}{\text{yr}}$$

Hence, a conservative estimate of the recurrence interval for a lightning strike on the reactor containment buildings is:

$$\frac{1}{0.931\ \text{flashes/yr}} = 1.07\ \text{years/flash}$$

2.3.1.2.2 Tornadoes and Severe Winds

Illinois ranks eighth in the United States in average annual number of tornadoes (Reference 11). Tornadoes occur with the greatest frequency in Illinois during the months of March through June. For the period 1916-1969, the publication "Illinois Tornadoes" (Reference 11) lists 62 tornadoes which occurred in the 7-county area (Will, Cook, DuPage, Kane, Kendall, Grundy, and Kankakee) surrounding and including the Braidwood Station site. Figure 2.3-1 shows the county distribution of tornadoes for the entire state for the same period of record. For Will County, the total number of tornadoes was 8, while for adjacent Grundy County the number of tornadoes was 3.

Tornadoes can occur at any hour of the day but are more common during the afternoon and evening hours. About 50% of Illinois tornadoes travel from the southwest to northeast. Slightly over 80% exhibit directions of movement toward the northeast through east. Fewer than 2% move from a direction with some easterly component (Reference 11).

The likelihood of a given point being struck by a tornado can be calculated by using a method developed by H. C. S. Thom (Reference 12). Thom presents a map of the continental United States showing the mean annual frequency of occurrence of tornadoes for each 1° square (latitude x longitude) for the period 1953-1962. For the 1° square (approximately 3600 mi^2 in area) containing the Braidwood Station site, Thom computed an annual average of 1.7 tornadoes. Assuming 2.82 mi^2 is the average area covered by a tornado (Reference 12), the mean probability of a tornado occurring at any point within the 1° square containing the Braidwood site in any given year is calculated to be .0014. This converts to a mean recurrence interval of 735 years. Using the same annual frequency but an average area of tornado coverage of 3.5 mi^2 (from Reference 11), the mean probability of a tornado occurrence is .0017.

More recent data (Reference 8) containing tornado frequencies for the period 1955-1967 indicate an annual tornado frequency of 3.3 for the 1° square containing the Braidwood site. This frequency, with Wilson and Changnon's average path area of 3.5 mi^2, results in an estimated mean tornado probability of .0033, with a corresponding mean return period of 305 years.

For the period 1970-1977, the NOAA publication "Storm Data" lists 46 tornadoes which have occurred in the seven-county area (Cook, Du Page, Grundy, Kane, Kankakee, Kendall, and Will) surrounding and including the Braidwood site. The majority of these tornadoes were short in length, narrow in width, and weak in intensity. However, seven of these tornadoes were severe enough to cause a large amount of damage in the area.

The most destructive tornado recorded during the period 1970-1977 in the vicinity of the Braidwood site occurred on March 12, 1976 between Northlake and Wilmette in Cook County. Damage included the twisting of the steel-beamed frame of an office building and the destruction of several homes.

A tornado which was nearly as destructive and likely more intense than the tornado described above occurred on June 13, 1976 near Lemont in Cook County. This tornado devastated an eight block area.

Based on the damage from the Lemont tornado of June 13, 1976, the maximum wind speed of the tornado was approximately 207-260 mph. The tornado path length extended 12 km with an average width of 1.6 km. A conservative estimate of the tornado path area is 19.4 km^2.

The above results were presented in order to provide a reasonable estimate of tornado probability without addressing the accuracy of the estimate. Because of the uncertainties in regard to tornado frequency and path area data, the annual tornado probability for the Braidwood site area is best expressed as being in the range of .0015 to .0030, with a mean tornado return period of 330 to 670 years. However, a conservatively high estimate can be taken to be .0033, with a corresponding mean return period of 305 years.

The following are the design-basis tornado parameters that were used for the Braidwood Station (Reference 13):

a. rotational velocity = 290 mph,

b. maximum translational velocity = 70 mph,

c. radius of maximum rotational velocity = 150 ft,

d. pressure drop = 3.0 psi, and

e. rate of pressure drop = 2.0 psi/sec.

The design wind velocity used for Seismic Category I structures at the Braidwood Station site is 85 mph considering a 100-year recurrence interval. For Seismic Category II structures, the governing design wind velocity used is 75 mph with a recurrence interval of 50 years. The design wind velocities for the 50-year and 100-year recurrence intervals are estimated from the analyses presented in Figures 1 and 2 of Reference 14. The vertical velocity distribution and gust factors employed for the wind loading are from Reference 14 for exposure type C (see Subsection 3.3.1).

2.3.1.2.3 Heavy Snow and Severe Glaze Storms

Severe winter storms which usually produce snowfall in excess of 6 inches and are often accompanied by damaging glaze are responsible for more damage in Illinois than any other form of severe weather, including hail, tornadoes, or lightning (Reference 15). These storms occur on an average of five times per year in the state. The state probability for one or more severe winter storms in a year is virtually 100%, while the state probability for three or more in a year is 87%. A typical storm has a median point duration of 14.2 hours. Point durations have ranged from 2 hours to 48 hours during the 61-year period of record 1900 to 1960 used in the severe winter storm statistical analyses (Reference 15). Data on the average areal extent of severe winter storms in Illinois show that they deposit at least 1 inch of snow over 32,305 mi^2, with more than 6 inches of snow covering 7,500 mi^2. Northeastern Illinois (including the Braidwood site) had 138 occurrences of a 6-inch snow or glaze damage area during the years 1900-1960. About 43 of those storms deposited more than 6 inches of snowfall in extreme southern Will County (the Braidwood site area).

Sleet or freezing rain occurs during the colder months of the year when rain falls through a shallow layer of cold air with a temperature below 0°C from an overlying warm layer of temperature above 0°C. The rain becomes supercooled as it descends through the cold air. If it cools enough to freeze in the air, it descends to the ground as sleet; otherwise, it freezes upon contact with the ground or other objects, causing glaze.

In Illinois during the 61-year period 1900-1960, there were 92 glaze storms defined either by the occurrence of glaze damage or by occurrence of glaze over at least 10% of Illinois. These 92 glaze storms represent 30% of the total winter storms in the period. The greatest number of glaze storms in 1 year was 6 (1951); in 2 years, 9 (1950-51); in 3 years, 10 (1950-52); and in 5 years, 15 (1948-52). In an analysis of these 92 glaze storms, Changnon (Reference 15) determined that in 66 storms, the heaviest glaze disappeared within 2 days; in 11 storms, 3 to 5 days; in 8 storms, 6 to 8 days; in 4 storms, 9 to 11 days; and in 3 storms, 12 to 15 days. Fifteen days was the maximum persistence of glaze.

Within the northern third of Illinois, eight localized areas received damaging glaze in an average 10-year period; the Braidwood site area averages about 4 days of glaze per year (Reference 15).

Ice measurements recorded in some of the most severe Illinois glaze storms are shown in Table 2.3-2 (Reference 15). The listing reveals that severe glaze storms depositing ice of moderate to large radial thickness may occur in any part of Illinois. An average of one storm every 3 years will produce glaze ice 0.75 inch or thicker on wires (Reference 15).

Strong winds during and after a glaze storm greatly increase the amount of damage to trees and power lines. In studying wind effects on glaze-loaded wires, the Association of American Railroads (Reference 16) concluded that maximum wind gusts were not as significant (harmful) a measure of wind damage as were speeds sustained over 5-minute periods. Moderate wind speeds (10-24 mph) occurring after glaze storms are most prevalent. Wind speeds of 25 mph or higher are not unusual, however, and there have been 5-minute winds in excess of 40 mph with glaze thicknesses of 0.25 inch or more (Reference 15). Specific glaze thickness data for the five fastest 5-minute speeds, and the speeds with the five greatest measured glaze thickness were measured in the after-storm periods of 148 glaze storms throughout the country during the period 1926-1937 and are shown in Table 2.3-3. Although these data were collected from various locations throughout the United States, they are considered to be applicable to design values for locations in Illinois. The roofs of safety-related structures are designed to withstand the snow and ice loads due to a winter probable maximum precipitation (PMP) with a 100-year recurrence interval antecedent snowpack. A conservative estimate of the 100-year return period snowpack weight of 28 psf (27 inches of snowpack) was obtained from the American National Standard building code requirements (Reference 14). The weight of the accumulation of the winter PMP from a single storm is 76 psf (14.6 inches of precipitable water, or about 146 inches of fresh snow), which was taken as the 48-hour PMP during the winter months from December through March (Reference 17). The design-basis snow and ice load is then 104 psf (see Subsection 2.4.2).

2.3.1.2.4 Ultimate Heat Sink Design

The ultimate heat sink (UHS) pond is designed to fulfill its purpose under the extreme environmental conditions set forth in NRC Regulatory Guide 1.27. The Peoria weather tape was used to evaluate the cooling capacity and evaporation losses of the UHS.

Meteorological data (January 1948 to August 1974) from the nearby National Weather Service station at Peoria, Illinois, were used in evaluating the performance of the cooling pond as an ultimate heat sink. Peoria data were used as the most representative of conditions at the Braidwood Station site. Since Peoria weather data were not available for January 1952 through December 1956, Springfield, Illinois, data were substituted for that period, as it was considered to be most representative of conditions at Peoria. Table 2.3-43 is provided to compare Springfield, Illinois data with similar Braidwood data to ensure that the Springfield data are the most representative available (for the 1952-56 period) of the Braidwood site. Wind speed, dry bulb temperature, dew-point temperature, and solar radiation data taken from the period June 8, 1952, to July 7, 1952, constituted the 30-day worst-case evaporation episode during the period January 1948 to August 1972. The mean dry bulb temperature, mean dew-point temperature, and mean wind speed recorded during these 30 days were 80.1°F, 66.3°F, and 10.1 mph, respectively.

A synthetic worst-case temperature period was constructed by using the worst 24-hour period weather data for the first day and the worst consecutive 30 days for the second to thirty-first days. The worst-case temperature data were again selected from the 308-month period of record of January 1948 to August 1974. For the worst 24 hours (July 19, 1964), mean dry bulb temperature, mean dew-point temperature, and mean wind speed were determined to be 80.3°F, 72.4°F, and 3.9 mph, respectively. For the worst 30 days (July 23, 1955, to August 21, 1955) the above mean conditions were determined to be 80.0°F, 67.9°F, and 7.9 mph, respectively. For details of ultimate heat sink design, see Subsections 2.4.11, 2.5.6 and 9.2.5.

2.3.1.2.5 Inversions and High Air Pollution Potential

Thirteen years of data (1952-1964) on vertical temperature gradient from Argonne (Reference 4) provide a measure of thermodynamic stability (mixing potential). Weather records from many U. S. stations have also been analyzed with the objective of characterizing atmospheric dispersion potential (References 18 and 19).

The seasonal frequencies of inversions based below 500 feet for the Braidwood Station site area are shown by Hosler as:

SEASON	% OF TOTAL HOURS	% OF 24-HOUR PERIODS WITH AT LEAST 1 HOUR OF INVERSION
Spring	30	68
Summer	31	81

SEASON	% OF TOTAL HOURS	% OF 24-HOUR PERIODS WITH AT LEAST 1 HOUR OF INVERSION
Fall	38	72
Winter	28	48

Since northern Illinois has a primarily continental climate, inversion frequencies are closely related to the diurnal cycle. The less frequent occurrence of storms in summer produces a larger frequency of nights with short-duration inversion conditions.

Holzworth's data give estimates of the average depth of vigorous vertical mixing, which gives an indication of the vertical depth of atmosphere available for mixing and dispersion of effluents. For the Braidwood Station region, the seasonal values of the mean maximum daily mixing depths (in meters) are:

	Mean Daily Mixing Depths	
Season	Morning	Afternoon
Spring	480	1500
Summer	320	1600
Fall	400	1200
Winter	470	610

When daytime (maximum) mixing depths are shallow, pollution potential is highest.

Argonne data are presented below in terms of the frequency of inversion conditions in the 5.5-foot to 144-foot layer above the ground as percent of total observations and in terms of the average duration of inversion conditions.

MONTH	INVERSION FREQUENCY	FIRST HOUR	FINAL HOUR
January	30.5%	5 p.m.	8 a.m.
April	33.1%	6 p.m.	6 a.m.
July	42.4%	6 p.m.	6 a.m.
October	48.4%	5 p.m.	7 a.m.

Nocturnal inversions begin at dusk and normally continue until daylight the next day. The inversion frequency for January at Argonne compares well with Hosler's winter value, and the fall season shows a maximum in both Argonne and Hosler's data. Fall also has the longest period of inversion conditions.

Holzworth has also presented statistics on the frequency of episodes of high air pollution potential, as indicated by low mixing depth and light winds (Reference 19). His data indicate that, during the 5-year period 1960-1964, the region including the Braidwood site experienced no episodes of 2 days or longer with mixing depths less than 500 meters and winds less than 2 m/sec. There were two such episodes with winds remaining less than 4 m/sec. For mixing heights less than 1000 meters and winds less than 4 m/sec, there were about nine episodes in the 5-year period lasting 2 days or more but no episodes lasting 5 days or more. Holzworth's data indicate that northern Illinois is in a relatively favorable dispersion regime with respect to low frequency of extended periods of high air pollution potential.

To help substantiate this statement, the 1977 air quality status for northern Illinois is provided in Tables 2.3-44 through 2.3-47. Provided are data for sulfur dioxide (SO_2) and total suspended particles (TSP) recorded at monitoring sites in northeastern Illinois as documented in the "Annual Air Quality Report 1977," Illinois Environmental Protection Agency, Springfield, Illinois.

2.3.2 Local Meteorology

2.3.2.1 Normal and Extreme Values of Meteorological Parameters

Onsite meteorological data available for the period of record January 1, 1974, through December 31, 1976 are summarized to describe the site meteorology. Meteorological data from Greater Peoria Airport (86 miles south-southwest of the Braidwood site), Argonne National Laboratory (ANL) (35 miles north-northeast of the Braidwood site), and the Dresden Nuclear Power Station meteorological tower (11 miles north-northwest of the Braidwood site) are used as the regional data for the Braidwood Station site. Whenever feasible, meteorological statistics for Peoria have been derived from the 3-hourly observations on magnetic tape per NCC Reference Manual TDF-14 for the 10-year period 1966-1975. The remaining Peoria data have been extracted from NOAA Local Climatological Data Summaries (1966-1975) (Reference 2). A 15-year (1950-1964) climatological summary compiled by ANL (Reference 4) is used as a comparative long-term data base. Some meteorological data from Dresden Nuclear Power Station (1974-1976) are used as the short-term data base for comparison with the onsite data.

2.3.2.1.1 Winds

Wind roses for the Braidwood Station site have been prepared from detailed onsite wind data for the period January 1974 through December 1976. The annual and monthly period-of-record wind roses for the 30-foot tower level are presented in Figures 2.3-2 through 2.3-14. The annual period-of-record wind rose for the

199-foot tower level is presented in Figure 2.3-15. The period-of-record wind roses and persistence of wind direction data for both tower levels are presented in Tables 2.3-4 through 2.3-6.

The annual surface wind rose for Peoria (1966-1975) is presented in Figure 2.3-16. Four seasonal wind roses for the 19-foot level of Argonne (1950-1964) are presented in Figures 2.3-17 through 2.3-20. Wind direction persistence data at Argonne and Peoria for the same period and levels are presented in Tables 2.3-7 and 2.3-8, respectively.

The 30-foot annual wind rose for the Braidwood site (Figure 2.3-2) shows that prevailing winds are from the south-southwest and south. The combined frequency of winds in these two sectors is approximately 21%. A large percentage of the winds from the prevailing directions has speeds greater than 3 m/sec. The frequency of winds from other directions is fairly evenly distributed. Calms occur during 2.25% of the year. The annual frequency distribution of wind directions at the 30-foot level (Figure 2.3-2) is similar to the frequency distribution at the 199-foot level (Figure 2.3-15), suggesting that the low-level wind direction is not affected very much by nearby topography or vegetation. The prevailing wind direction at the 199-foot level is from the south-southwest. The frequency of higher wind speeds is greater for all directions, and winds with a westerly component are slightly more frequent. Calms occur 0.25% of the time at the 199-foot level, less frequently than at the 30-foot level.

The 30-foot onsite monthly wind roses (Figures 2.3-3 through 2.3-14) indicate that the colder months of November through March have a relatively high frequency of winds from the south to south-southwest and the west to northwest. April and May exhibit a greater frequency of winds from the south to southwest and the north-northeast clockwise through east-northeast. The prevailing winds during the summer blow from the south to southwest, while a secondary frequency maximum of winds is found in the north-northeast through east-northeast sectors. Winds during September and October blow more frequently from the north and south than the east and west. The months of January through April experience the greatest frequency of wind speeds higher than 7.0 m/sec. Light winds, with speeds less than 1.5 m/sec, occur most frequently from July through September. Calms are least frequent from January through April and most frequent from July through September.

The 30-foot and 199-foot level persistence of wind direction at the Braidwood Station site (1974-1976) shows that the majority of cases at both levels are short-period persistences of less than 3 hours (Tables 2.3-5 and 2.3-6). The longest persistence at the 30-foot level over the 3-year period was from 25 to 27 hours for winds from the south and south-southwest. The 199-foot level data indicate that a north-northwest wind once blew for 34 to 39 consecutive hours.

In general, winds persist for a greater length of time at the 199-foot level. Persistent calms are rare at this level, with the longest persistence of calms (4 to 6 hours) occurring once during the period of record. In contrast, the 30-foot level had 35 cases of calms which persisted for 4 or more hours, with a maximum persistence of about 15 hours.

The surface annual wind rose for Peoria for the long-term period of record 1966 through 1975 (Figure 2.3-16) shows that the prevailing wind is from the south and occurs with an annual frequency of over 17%. The directional distribution of wind for the remaining 15 sectors is fairly even, with a frequency for each sector between 3% and 8%. The greatest frequency of strong winds (speeds greater than 6.3 m/sec) occurs with winds from the west, the west-northwest, and south sectors. The north-northeast clockwise through southeast sectors have a comparatively large percentage of light to moderate winds (speeds less than 6.3 m/sec). The frequency of calms at Peoria is 3.55%.

A comparison of the Braidwood site 30-foot level annual wind rose with the Peoria surface annual wind rose indicates that they are in general agreement, indicating that the Braidwood site wind data are representative of the surrounding region. One of the differences is that the wind direction frequency distribution has a pronounced peak in the south sector at Peoria, while the peak at the Braidwood site is less pronounced, with the prevailing wind distributed among the south and south-southwest sectors. Another difference is that the frequency of calms at Peoria (3.55%) is greater than that at the Braidwood site (2.25%), which is partially due to the fact that the surface wind at Peoria is measured at a lower level (20 feet).

Wind direction persistence data for Peoria (Table 2.3-7) indicate that the prevailing southerly winds persist for the longest period; a persistence of over 45 hours occurred four times during the 10-year period of record. Winds from the south persisted for more than 24 hours 76 times, compared with a total of only 18 times for all other wind directions. The maximum persistence of calms was 15 hours at Peoria.

Seasonal wind roses for Argonne (Figures 2.3-17 through 2.3-20) indicate that in winter winds blow most frequently from the southwest clockwise through the northwest with a maximum frequency from the west. The spring wind rose shows that the wind directions are more evenly distributed, and that winds most frequently blow from a generally southwesterly and northeasterly direction. The summer wind rose is similar to the spring wind rose, except that the frequency of wind direction is more pronounced along a northeast to southwest axis, and the wind speeds are generally less. Winds from the southwest prevail during the fall, and there is a return to a greater frequency of west winds. All winds with speeds less than 3 mph (1.3 m/sec) are classified as calm at Argonne. Calms are least common in spring and most common in summer.

Wind direction persistence data for Argonne (1950-1964) given in Table 2.3-8 indicate that southwesterly and south-southwesterly winds persist for the longest period of time at both the 19-foot and 150-foot levels. The maximum persistence of calms at Argonne was 12 hours.

Argonne lies approximately 20 miles west-southwest of Lake Michigan, while the Braidwood site is approximately 50 miles from the lake. A more frequent occurrence of northeasterly winds at Argonne during the spring and summer as compared with the Braidwood site wind statistics indicates that Argonne is influenced on occasion by lake effects from Lake Michigan, hence long-term Argonne wind data are less representative of conditions at the Braidwood site than long-term Peoria wind data.

2.3.2.1.2 Temperatures

Short-term monthly and annual data for average and extreme temperatures for the Braidwood site (1974-1976) are compared in Table 2.3-9 with those for Peoria (1973-1976) and Dresden Nuclear Power Station (1974-1976) in order to show the representativeness of the Braidwood site data for the region.

Long-term monthly and annual average and extreme temperature data for Peoria (1966-1975) and Argonne (1950-1964) are compared in Table 2.3-10 with short-term (1974-1976) temperature statistics measured at the 30-foot level of the Braidwood site meteorological tower.

Temperatures are measured at the 5.5-foot level at Argonne. Temperature measurements at Peoria are made at the National Weather Service standard height of 4.5 feet above the surface. The 30-foot level measurement for lower level temperature at the Braidwood site and the 35-foot level measurement for lower level temperature at Dresden Nuclear Power Station are essentially in accordance with the requirement of NRC Regulatory Guide 1.23.

The short-term average and extreme temperature data (Table 2.3-9) at the Braidwood Station site (1974-1976), Dresden Nuclear Power Station (1974-1976), and Peoria (1973-1975) show that the 3-year average temperature at the Braidwood site and at Dresden are in good agreement; the stations have a 3-year average temperature of 49.7°F and 49.6°F, respectively.

The highest temperature reported at Peoria during the period January 1973 to December 1975 was 97°F, while the lowest temperature was -18°F. Extremes at the Braidwood Station site (1974-1976) were 96.4°F and -13.0°F, while extremes during the same period at Dresden were 95.0°F and -14.5°F. The temperature extremes for the Braidwood site and for Dresden are quite similar, suggesting that they are representative of the temperature extremes experienced in the region surrounding the Braidwood site. Monthly maxima and minima at Peoria show a substantial departure from similar data at the Braidwood and Dresden sites for a few months (e.g., November). These differences are attributed to a difference in measuring height and a different period of record.

Long-term temperature averages for Peoria (1966-1975) and Argonne (1950-1964) were 49.8°F and 47.7°F, respectively (Table 2.3-10). The short-term temperature average at the Braidwood site (1974-1976) was 49.7°F. Long-term temperature extremes for Peoria were 102°F and -18°F, and for Argonne, 101°F and -20°F. Extremes for the Braidwood site were 96.4°F and -13.0°F. The larger temperature ranges at Peoria and Argonne are attributed, in part, to their lower measuring height and to their longer period of record.

Table 2.3-11 presents the average daily maximum and minimum temperatures at Peoria, Illinois (1966-1975). The mean daily diurnal temperature variation ranges from about 11°F in December to about 19°F in June and October.

2.3.2.1.3 Atmospheric Moisture

Data for the atmospheric moisture parameters measured at the 35-foot level of the Dresden Nuclear Power Station meteorological tower (1975-1976) are presented as representative of short-term atmospheric moisture conditions at the Braidwood site. Onsite atmospheric moisture parameter measurements are not presented due to a low data recovery rate. Long-term offsite atmospheric moisture data for Peoria (1966-1975) and Argonne (1950-1964) are presented and compared with the Dresden data.

2.3.2.1.3.1 Relative Humidity

The relative humidity for a given moisture content of the air is defined as the ratio of the actual mixing ratio of water vapor to that which would exist at saturation at the same temperature. The diurnal variation of relative humidity for a given moisture content of the air is inversely proportional to the diurnal temperature cycle. A maximum in relative humidity usually occurs during the early morning hours, while a minimum is typically observed in midafternoon.

Relative humidity data for the 35-foot level at Dresden (1975-1976) are presented in Table 2.3-12. The annual average relative humidity is 71.2%. The maximum relative humidity observed at Dresden during the period is 100% for all months except April (98.8%), while the minimum relative humidity observed during the 2-year period of record is 17.7%, for April.

Long-term relative humidity data for Peoria (1966-1975) and Argonne (1950-1964) are presented in Table 2.3-13. The annual average relative humidity is 71.2% at Peoria and 74.9% at Argonne. Monthly average relative humidities are also higher at Argonne, especially during the late winter and early spring months. These differences in average relative humidity are primarily due to the close proximity of Argonne to Lake Michigan. The Peoria data are considered to be more representative of relative humidity conditions at the Braidwood site, since the influence of Lake Michigan upon relative humidities at both locations is minimal as compared with the influence of the lake at Argonne.

The maximum relative humidity observed at Peoria is 100% for all months, while the minimum relative humidity observed during the period of record is 14%, observed in the months of March, April, and October. These extremes are comparable with the short-term extremes observed at Dresden (100% and 17.7%).

The annual average daily maximum and minimum relative humidities at Peoria are about 85% and 55%. The monthly average diurnal relative humidity range is greatest in May and least in December.

2.3.2.1.3.2 Dew-Point Temperature

The dew-point temperature is defined as the temperature to which air must be cooled to produce saturation with respect to water vapor, with pressure and water vapor content remaining constant.

Dew-point temperature data for Dresden (1975-1976) are presented in Table 2.3-14. The 2-year average dew-point temperature at Dresden is 39.5°F. Monthly average dew-point temperatures at Dresden range from 18.9°F in January to 62.2°F in July. The maximum dew-point temperature during the period was 77.1°F, while the minimum dew-point temperature was -13.7°F.

Long-term dew-point temperature data for Peoria and Argonne are presented in Table 2.3-15. The annual average dew-point temperature at Peoria (40.3°F) and Argonne (38.7°F) are generally comparable with the short-term average at Dresden (39.5°F).

Monthly average dew-point temperatures at Peoria range from 15.8°F in January to 62.4°F in July. The maximum dew-point temperature during the 10-year period is 79°F, while the minimum dew-point temperature is -27°F.

The annual average daily maximum and minimum dew-point temperatures at Peoria (1966-1975) are 45.1°F and 35.4°F. The maximum average diurnal variation in dew-point temperature is 13.3°F in January, while the minimum average diurnal variation is 7.2°F in August and September.

2.3.2.1.4 Precipitation

Precipitation data are available from the Braidwood site, Peoria, and Argonne, and serve to indicate the precipitation characteristics of the Braidwood Station site.

2.3.2.1.4.1 Precipitation Measured as Water Equivalent

Monthly precipitation totals at the Braidwood site and Peoria for the short-term period of record January 1, 1974, through December 31, 1976, are compared in Table 2.3-16. The precipitation totals at both stations for each month during the period of record shown are generally similar, indicating the representativeness of the Braidwood site data for the region. The heaviest monthly rainfalls occurred during late spring or early summer months. The maximum monthly rainfall for each location during the 3-year period was 6.61 inches at the Braidwood site (May 1974) and 11.69 inches at Peoria (June 1974). The lightest monthly rainfalls generally occurred during the fall or winter months. The minimum monthly rainfall was 0.02 inch at Braidwood (September 1975) and 0.38 inch at Peoria (December 1976).

Long-term normal and extreme monthly and annual precipitation totals (water equivalent) for Peoria (1966-1975) and Argonne (1950-1964) are presented in Table 2.3-17. The maximum and minimum monthly precipitation totals recorded for Peoria (1966-1975) are 11.69 inches during June 1974, and 0.56 inches in February 1969, respectively. The maximum and minimum monthly precipitation levels recorded for Argonne (1950-1964) are 13.17 inches during September 1954, and 0.03 inches during January 1961, respectively. In general, more than twice as much precipitation falls during the warmer summer months than during the colder winter months, due to increased convective storm activity and the higher specific humidity of warm air, both of which occur during the warmer months. The annual average precipitation at Peoria was 37.80 inches, while at Argonne, the average precipitation was 31.49 inches per year. The differences in precipitation averages and extremes measured at Peoria and Argonne are primarily due to the differences in geographical location and to the nonoverlapping period of record.

The average number of hours of precipitation per month at Peoria (1966-1975) and Argonne (1950-1964) are as follows:

MONTH	PEORIA	ARGONNE
January	125	89
February	108	75
March	118	99
April	88	94

MONTH	PEORIA	ARGONNE
May	78	65
June	48	55
July	37	58
August	40	37
September	64	40
October	72	55
November	106	73
December	172	95
Annual Average	1,054	834

The above table indicates that there are nearly three times as many hours in winter with precipitation than in summer. The data from the above table and the Peoria average monthly rainfall data (Table 2.3-17) reflect the fact that summer precipitation is generally heavy, showery, and brief, while winter precipitation is less intense, steady, and occurs over a longer period of time. These data indicate that Argonne's approximately 20% fewer hours of precipitation is consistent with Argonne's approximately 20% smaller annual average precipitation totals.

Table 2.3-18 presents the monthly and annual joint frequency distribution of wind direction and precipitation for Peoria (1966-1975). During the winter, precipitation occurrences are relatively evenly distributed among all wind directions, with a somewhat higher frequency of occurrences with winds from the north clockwise through south. During the summer, precipitation occurs most frequently with winds from the south. On an annual basis, precipitation occurs most frequently (1.6% of the time) with the prevailing southerly winds.

Table 2.3-19 presents maximum precipitation (water equivalent) for specified time intervals at Argonne (1950-1964) and the 24-hour maximum precipitation totals at Peoria (1966-1975). The maximum 1-hour total recorded at Argonne is 2.20 inches (June 1953), while the maximum 48-hour total is 8.62 inches (October 1954). The 24-hour maximum precipitation of 4.44 inches at Peoria occurred in June. The largest maximum short-period (less than 24 hours) totals are seen to occur during the summer months when heavy convective activity is at its peak, while maximum precipitation totals for winter are less than for the other seasons.

2.3.2.1.4.2 Precipitation Measured as Snow or Ice Pellets

Monthly average, monthly maximum, and 24-hour maximum totals of snow and/or ice pellet precipitation (in inches) for Peoria (1966-1975) are presented in Table 2.3-20. Annual Peoria totals average 24.1 inches. The greatest monthly average total is 6.3 inches for January. The maximum monthly total for the entire period is 18.9 inches, recorded in December. The maximum 24-hour total is 10.2 inches, also recorded in December.

The extreme values of precipitation (including snow) recorded at Peoria, for a period longer than 1966-1975 are presented in Table 2.3-1. Also, the extreme values of precipitation recorded at Aurora and Kankakee longer than 10 years are presented in Table 2.3-48. The maximum monthly total for the entire period is 14.86 inches at Aurora and 10.69 inches at Kankakee. The maximum 24-hour total is 10.48 inches at Aurora and 8.43 inches at Kankakee. Therefore, Tables 2.3-1, 2.3-20, and 2.3-48 provide a record of the extreme values of precipitation at five separate long-term stations in the vicinity of the Braidwood site.

2.3.2.1.5 Fog

Fog is an aggregate of minute water droplets suspended in the atmosphere near the surface of the earth. Fog types are generally coded as fog, ground fog, and ice fog in observation records. According to international definition, fog reduces visibility to less than 0.62 mile (Reference 7). Observing procedures by the National Weather Service define ground fog as that which hides less than six-tenths of the sky and does not extend to the base of any clouds that may lie above it (Reference 7). Ice fog is composed of suspended particles of ice. It usually occurs in high latitudes in calm clear weather at temperatures below -20°F and increases in frequency as temperature decreases (Reference 7).

Fog forms when the ambient dry bulb temperature and the dew-point temperature are nearly identical or equal. The atmospheric processes by which these temperatures become the same and fog occurs are either by cooling the air to its dew point or by adding moisture to the air until the dew point reaches the ambient dry bulb temperature. This latter process results in the formation of evaporation fog and is of particular interest with respect to cooling facility operation at power generation stations.

Cooling facility fog generally occurs when atmospheric conditions are conducive to natural fog formation. Natural processes such as radiational cooling during relatively calm nights or the advection of moist air over a cooler land surface are generally contributing factors. Thus the previous summary of natural fog occurrence is important in the understanding of the potential fogging problems at the Braidwood site.

Table 2.3-21 presents the frequency and persistence of fog at Peoria (1966-1975). Onsite data are not available to assess the fog characteristics at the Braidwood site. The annual average number of hours with fog is 1162 hours per year at Peoria. The month of December has the highest average number of hours with fog (189 hours), meaning that fog normally occurs during approximately 25% of the 744 hours in December. The month of June averages only 37 hours of fog per month.

Most fog occurrences have a relatively short duration. During the 10-year period, over 60% of the periods of fog persisted for less than or equal to 6 hours, while only about 0.5% of the periods of fog persisted for more than 24 hours. This trend is more pronounced in the warmer part of the year when most fog is nocturnal radiation fog which dissipates in the morning. In June, the maximum fog persistence was 18 hours. In contrast, there were 32 cases in December where fog persisted for more than 18 hours. All periods of fog lasting more than 46 hours occurred during December, January, February and March.

Table 2.3-22 presents the seasonal and annual distribution of fog by hour of the day for Peoria. The annual data show that most fog tends to occur during the latter part of the night and the early morning. An annual average of 66.8% of all fog occurs from midnight until the 9:00 a.m. observation. Fog occurrences are most evenly distributed throughout the day during winter, when days are short and solar insolation is weak; winter has the greatest number of hours of fog as well as the most persistent fog. Spring and fall both have a greater percentage of fog occurrences late at night and early in the morning. Summer shows the most extreme diurnal variation of fog occurrence, with about 45% of all fog occurring at the 6:00 a.m. 3-hourly observation. Only 12.3% of summertime fog occurs from noon until the 9:00 p.m. observation. The Peoria surface weather observations are taken at Greater Peoria Airport on relatively flat land above the Illinois River. Some moisture addition from the river and wet valley areas to the atmosphere occurs, and probably causes a somewhat higher frequency of fog at Greater Peoria Airport than in areas located far from the river. The Braidwood site is located on flat land, relatively far from a major river. Hence the frequency of fog at the Braidwood site is expected to be somewhat less than at Greater Peoria Airport.

2.3.2.1.6 Atmospheric Stability

Onsite differential temperature data between the 30-foot and 199-foot tower levels at the Braidwood site were used to estimate stability. Monthly and annual Pasquill stability class frequencies for the period of record January 1974 through December 1976 at the Braidwood site are presented in Table 2.3-23. Table 2.3-24 presents the persistence of Pasquill stability classes for the same period of record.

Examination of Table 2.3-23 indicates that the neutral or D stability class occurs 32.9% of the time, while the slightly stable or E stability class occurs with a slightly higher frequency of 37.7%. The frequencies of the stability classes on either side of D and E taper off sharply. Such results are inherent in this classification scheme.

The combination of low wind speeds, a constant wind direction, and a stable atmosphere produces the worst atmospheric dispersion conditions. Listed below is the longest persistence of one wind direction and of calms occurring during each stability class at the Braidwood site 30-foot level:

Pasquill Stability Class	Longest Persistence in One Wind Direction	Longest Persistence During Calms
A	11 hours (NNW)	1 hour
B	7 hours (NW)	1 hour
C	5 hours (W)	1 hour
D	19 hours (W)	4 hours
E	26 hours (S)	9 hours
F	13 hours (NNW)	6 hours
G	9 hours (SSE)	8 hours

Joint frequency distributions of wind direction and wind speed for each Pasquill stability class for the 30-foot and 199-foot levels at the Braidwood site (1974-1976) are presented in Tables 2.3-25 and 2.3-26. These data are used for the long-term diffusion estimates presented in Subsection 2.3.5.

The joint frequency of A stability and calms at the 30-foot level is 0.00%; B and calms 0.01%; C and calms 0.02%; D and calms, 0.18%; E and calms, 0.52%; F and calms, 0.82%; and G and calms, 0.79%. Although calms occur most frequently during stable conditions (Stability classes E, F, and G), their joint occurrence was only 2.13% of the 3-year period of record.

Figure 2.3-21 compares the short-term vertical temperature gradient histograms for Braidwood (1974-1976) and Dresden (1974-1976). The following data summarized from Figure 2.3-21 compare the Braidwood frequencies of occurrence (in percent) of each stability class with those of Dresden:

Stability Class	Braidwood	Dresden
A	2.92	9.92
B,C	8.05	8.60
D	32.20	28.53
E	38.30	33.18
F	12.38	15.00
G	6.15	4.76

As indicated above, there are some differences in the frequency distribution of stability classes between the Braidwood and Dresden sites (e.g., stability Class A occurs with a frequency of 2.92% at Braidwood, while at Dresden it occurs with a frequency of 9.92%). However, Figure 2.3-21 shows that the vertical temperature gradient histograms for both sites are similar. Both histograms approximate a Gaussian distribution, with a skew (or tailout) toward the larger positive (more stable) gradients. The slightly taller and narrower Braidwood peak can be expected, since the Braidwood upper temperature measurement level is 49 feet higher than Dresden's 150-foot level. The largest diurnal temperature variations occur near the ground due to surface heating during the day and radiative cooling at night.

Long-term joint frequency distributions of wind direction and wind speed for each Pasquill stability class at Peoria (1966-1975) are summarized in Table 2.3-27. Pasquill stability class frequencies at Peoria are derived from surface observations (1966-1975) using the criteria established by Pasquill (Reference 20). The resulting Peoria stability class frequencies (in percent) as compared with the shorter term Braidwood (1974-1976) stability class frequencies are shown below:

Stability Class	Braidwood	Peoria
A	2.92	0.29
B,C	8.05	15.03
D	32.20	60.81
E	38.30	10.39
F	12.38	9.49
G	6.15	3.99

The long-term Peoria data show a pronounced peak in the frequency distribution at neutral Class D of 60.81%. A comparison of these data with the Braidwood data shows that the Peoria frequencies are significantly higher for the C and D classes, and significantly lower for the A and E classes. Estimation of stability classifications from surface data, while useful when no other data exist, can lead to significant biases in stability frequency. This bias is generally toward neutral conditions and away from extremes of stability and instability.

Table 2.3-28 presents Pasquill stability class persistences for Peoria (1966-1975). Neutral stability Class D persists for a substantially larger number of consecutive occurrences of 3-hourly observations than any other stability class. The largest number of consecutive occurrences of 3-hourly observations of each stable stability Class E, F and G are as follows: 5 on 9 occasions for stability Class E; 5 on 8 occasions for stability Class F; and 4 on 18 occasions for stability Class G.

2.3.2.2 Potential Influence of the Plant and Its Facilities On Local Meteorology

Of singular importance as a factor affecting the local meteorology near the site is the presence of the cooling pond south of the plant for waste heat dissipation. The overall dimensions of the pond are approximately 2 miles by 3 miles long, with the major axis oriented in the northeast-southwest direction. Therefore, a considerable amount of contact time is available for an overlying air mass to be influenced by the heat and moisture dissipated from the cooling pond, particularly in the case of the northeasterly and southwesterly winds. The influence of the cooling pond on the local meteorology will be pronounced during the winter season, when the temperature differential between the pond and the air mass is a maximum, and during certain climatological conditions when the difference between the saturation vapor pressure and actual vapor pressure of the atmosphere is very small.

An air mass flowing over a surface of certain thermal properties is disturbed if the nature of the surface changes. An analysis presented by Godske et al. (Reference 21) for a stable atmosphere (temperature increases with height at a rate 20°C per 100 meters for the first 10 meters of height and at a rate of 2.0°C per 100 meters up to 80 meters in height) indicated that a slight modification of the air mass could extend to a height of 120 meters and persist after leaving the pond for a distance 2.5 times the length of the pond. A major change in air mass amounting to 40% of the initial temperature difference between the air and the pond could extend to heights of 60 meters and downwind distances beyond the pond boundary equal to 0.2 times the length of the pond. For the Braidwood Station site, this distance is approximately 0.6 mile.

It is expected that humidity will be increased in regions within a few hundred meters of the pond's shoreline and that additional periods of fog will be experienced, especially on the eastern side of the pond, which is the prevailing downwind area during the winter season. With the occurrence of below-freezing temperatures during the winter months, the fog will form hoarfrost and deposit rime ice when coming in contact with vertical surfaces.

Areas of potential concern due to additional fogging and icing include the town of Godley, located about 2000 feet from the northwest corner of the cooling pond, the switchyards east of the plant, and state roads 53 and 129, which pass within 2000 feet of the northwest edge of the cooling pond. In order to assess the fogging and icing potential, estimates of the ground-level concentrations of water vapor were made at these locations with the cooling pond in operation. These values were compared to the ambient moisture saturation values to determine the frequency and duration of increased fogging and icing.

The potential impacts of fogging and icing resulting from cooling pond operation at Braidwood Station were assessed using an atmospheric dispersion model and a moisture evaporation formula. An infinite line source dispersion model was used to describe the dispersion process. The basis of this model is an equation derived by Sutton (see Reference 23). The source term in the dispersion equation is a function of water vapor flux and cooling pond length.

Evaporation flux from the pond surface was calculated by using the Lake Colorado City formula (Reference 22) and estimated cooling pond temperatures. The transport and dispersion of water vapor were estimated using Sutton's equation (Reference 23) that describes dispersion from an infinite crosswind source. The source term used in the dispersion equation was the flux of water vapor times the distance the air mass traversed over the water. The 15-year summary of temperature and humidity statistics for the weather station at Argonne National Laboratory (Reference 4) was used as the ambient conditions to calculate the water vapor concentrations at the locations of interest. Results of the above analysis indicate that slight increases in atmospheric moisture are expected at the above-listed locations due to the operation of the cooling pond, and that the resulting increases in fogging or icing (less than 1 hour per year) are small compared with the natural occurrence of fog and ice. Except for some localized effects in the immediate vicinity of the pond, it is concluded that slight increases in fogging or icing will not be a significant nuisance.

The cooling pond is expected to modify dispersion conditions in a manner that enhances mixing in the first 60 to 100 meters of the atmosphere. This effect is important with ground-level releases. It is quite likely that the added mixing could be equivalent to shifting the diffusion regimes one whole stability class in the case of stable atmospheres. Unstable atmospheres will hardly be affected. For northeasterly and southwesterly winds which experience maximum travel over waters, dispersion conditions more favorable than indicated by regional meteorology could exist at downwind distances up to 1.3 times the length of the overwater trajectory due to pond effects. In the evaluation of χ/Q estimates discussed in Subsections 2.3.4 and 2.3.5, the influence of the pond on diffusion climatology was not included.

2.3.2.3 Topographical Description

Figure 2.3-22 is a topographic map showing the area surrounding the Braidwood site. Figure 2.3-23 shows the topographic cross section in each of the 16 compass point directions radiating from the site. The plant, at an elevation of approximately 600 feet above MSL, will be at one of the highest points within a 5-mile radius. The lowest points within 5 miles of the site are about 550 feet above MSL. Terrain in the vicinity of the plant falls off except in the northeast, east-northeast, east, east-southeast, southeast, and south-southeast directions (Figure 2.3-23, Sheet 3. The slope from the plant site to the lower points is gradual.

No significant effect of topography on atmospheric dispersion is anticipated due to the flat nature of the area. Accordingly, no topographic factors have been included in the dispersion calculations (Subsections 2.3.4 and 2.3.5).

2.3.3 Onsite Meteorological Measurements Program

The meteorological tower is located as shown on Figure 2.1-5.

The meteorological measurements program consists of monitoring wind direction, wind speed, temperature, and precipitation. Two methods of determining atmospheric stability used are:

a. delta T (vertical temperature difference) is the principal method, and

b. sigma theta (standard deviation of the horizontal wind direction) is available for use when delta T is not available.

These data, referenced in ANSI/ANS 2.5 (1984), are used to determine the meteorological conditions prevailing at the plant site. The meteorological program includes site-specific information on instrumentation and calibration procedures. The meteorological program meets the requirements of the Offsite Dose Calculation Manual.

The meteorological tower is equipped with instrumentation that conforms with the system accuracy recommendations in Regulatory Guide 1.23 and ANSI/ANS 2.5 (1984). The equipment is placed on booms oriented into the generally prevailing wind at the site.

Equipment signals are transmitted to an instrument shack with controlled environmental conditions. The shack at the base of the tower houses the recording equipment, signal conditioners, etc., used to process and retransmit the data to the end-point users.

Recorded meteorological data are used to generate wind roses and to provide estimates of airborne concentrations of gaseous effluents and projected offsite radiation dose. Instrument calibrations and data consistency evaluations are performed routinely to ensure maximum data integrity. The data recovery objective is to attain better than 90% from each measuring and recording system. Data storage and records retention are also maintained in compliance with ANSI/ANS 2.5 (1984)

Pages 2.3-28 through 2.3-32 have been deleted intentionally.

2.3.4 Short-Term (Accident) Diffusion Estimates

2.3.4.1 Objective

Conservative estimates of the local atmospheric dilution factors (χ/Q) and their 5% and 50% probability levels for the Braidwood Station site have been made. These estimates were made for the minimum exclusion area boundary (the minimum exclusion area boundary distances are provided in Table 2.3-49) and the outer boundary of the low population zone (LPZ) for each of the 16 cardinal directions. Estimates were made for time periods from 1 hour to 2 hours for the minimum exclusion area boundary and from 8 hours up to 30 days for the outer boundary of LPZ, utilizing onsite meteorological data (36-month period of onsite data recorded from January 1, 1974, through December 31, 1976).

Estimates of atmospheric diffusion (χ/Q) at the Exclusion Area Boundary (EAB) and the outer boundary of the Low Population Zone (LPZ), calculated for the regulated short-term (accident) time-averaging periods of 0-2 hrs, 2-8 hrs, 8-24 hrs, 1-4 days, and 4-30 days were also performed in support of Alternative Source Term (AST) implementation.

2.3.4.2 Calculations (For use with TID-14844 based dose analyses)

Calculations of the short-term atmospheric dilution factors (χ/Q) for the Braidwood Station site were performed using Gaussian plume diffusion models for ground-level concentrations resulting from a continuously emitting source. To be conservative, the effluent release level was assumed to be at ground level, and total reflection of the plume was assumed to take place at the ground surface; i.e., there is no deposition or reaction at the surface.

Hourly χ/Q values were calculated using a centerline diffusion model for time periods up to 8 hours and a sector-average diffusion model for time periods longer than 8 hours. A building wake correction factor that did not exceed a maximum of 3.0 was used in the centerline model to account for additional dilution due to wake effect of the reactor building. No credit was given for additional building wake dilution for the sector-average model. Mathematical expressions of the models are as follows:

a. For time periods up to 8 hours:

$$\chi / Q = \frac{1}{\pi u \sigma_y \sigma_z + cAu} \qquad (2.3\text{-}1)$$

$$= \frac{1}{D_b} \frac{1}{\pi u \sigma_y \sigma_z}$$

b. For time periods greater than 8 hours:

$$\chi / Q = \frac{2.032}{\sigma_z u \chi}$$

where:

χ/Q = ground-level relative concentration (sec/m^3),

u = mean wind speed (m/sec),

σ_y = horizontal diffusion parameter (m),

σ_z = vertical diffusion parameter (m),

c = empirical building shape factor,

A = reactor building minimum cross section (m^2),

D_b = building wake correction factor $(1 + \frac{CA}{\pi \sigma_y \sigma_z})$,

χ = distance from release point to receptor, and

2.032 = $\sqrt{2/\pi}$ ÷ the width in radians of a 22.5° sector.

Meteorological data input used were concurrent hourly mean values of wind speed and wind direction measured at the 30-foot level and Pasquill stability class determined by the measured vertical temperature difference (ΔT) between 30-foot and 199-foot levels.

Both σ_y and σ_z are functions of downwind distance from the point of release to a receptor and the Pasquill stability class. The numerical values of σ_y and σ_z for Pasquill stability classes A through F were digitized from Gifford's graphs (Reference 20). The values of σ_z for Pasquill stability classes A and B have been cut off at 1000 meters. The values of σ_y and σ_z for Pasquill stability class G were determined from σ_y and σ_z for Pasquill stability class F using the following equations:

$$\sigma_y(G) = \frac{2\sigma_y(F)}{3}$$

$$\sigma_z(G) = \frac{3\sigma_z(F)}{5} \qquad (2.3\text{-}3)$$

A building shape factor of 0.5 and building minimum cross section of 2700 m^2 were used to determine the building wake correction factor.

For calm wind conditions a wind speed of 0.17 m/sec, one-half of the threshold speed, was assigned, and computed χ/Q values for calm wind conditions were assigned to wind direction for the previous hour with a wind speed greater than the threshold speed.

From these hourly χ/Q values, mean values were computed for sliding time period "windows" of 2, 8, 16, 72 and 624 hours for each of 16 cardinal directions. These intervals correspond to accident time periods of 0-1 hour, 0-2 hours, 0-8 hours, 8-24 hours, 1-4 days, and 4-30 days. For each time interval used, the maximum χ/Q value in each sector was identified, and the mean χ/Q in each sector was computed. The cumulative frequency distribution for each of the individual time periods was then prepared, and from this distribution the fifth and fiftieth percentile χ/Q values were estimated for each of the 16 cardinal sectors.

Cumulative frequency distributions of χ/Q values and 5% and 50% probability levels of χ/Q at the minimum exclusion area boundary distance of 485 meters (minimum actual site boundary distance) are presented in Tables 2.3-29 through 2.3-31 for accident time periods of 0-1 hour and 0-2 hours. Cumulative frequency distributions of χ/Q values and the maximum, 5%, and 50% probability levels of χ/Q values at the outer boundary of the LPZ are presented on Tables 2.3-32 through 2.3-38 for accident time periods of 0-8 hours, 8-24 hours, 1-4 days, and 4 to 30 days.

2.3.5 Long-Term (Routine) Diffusion Estimates (For TID-14844 based dose analyses)

2.3.5.1 Objective (For TID-14844 based dose analyses)

Realistic estimates of annual average atmospheric dilution factors (χ/Q) for effluents released routinely from the 200-foot Braidwood vent stack have been made. These estimates are made for site boundary distances and for various distances out to 50 miles (80.5 km) for each of the 16 cardinal directions, utilizing onsite meteorological data (36-month period of onsite data recorded from January 1, 1974, through December 31, 1976).

2.3.5.2 Calculations (For TID-14844 based dose analyses)

Annual average atmospheric dilution factors (χ/Q) for the Braidwood Station site are calculated using the sector-average Gaussian plume diffusion model (constant mean wind direction model) modified to account for various modes of effluent release according to the recommendations of Regulatory Guide 1.111.

The effects of spatial and temporal variations in airflow in the region of the Braidwood site are not described by the constant mean wind direction model (used in calculating the Braidwood χ/Q values) since this model uses single-station meteorological data to represent diffusion conditions within the vicinity of a site. Regulatory Guide 1.111, "Methods for Estimating Atmospheric Transport and Dispersion of Gaseous Effluents in Routine Releases from Light-Water-Cooled Reactors" (July 1977), recommends that if a constant mean wind direction model is used, airflow characteristics within 50 miles of the site should be examined to determine the spatial and temporal variations of atmospheric transport and diffusion.

Recirculation of airflow and wind directional biases during periods of prolonged atmospheric stagnation are the primary variations in atmospheric transport and diffusion conditions to be considered for the Braidwood site. Airflow is not inhibited by topography in the vicinity of Braidwood due to the relatively flat nature of the terrain. A detailed topographic description is presented in Subsection 2.3.2.3. Regional airflow is dominated by large-scale (synoptic) weather patterns. A summary of these climatological patterns is in Subsection 2.3.1. Figures 2.3-24 through 2.3-26 are wind roses for locations near the Braidwood site. The similarity of wind direction distribution from location to location indicate the common wind regime representative of the area.

A combination of low wind speeds, a constant wind direction, and a stable atmosphere produces the worst atmospheric dispersion conditions. Under very stable conditions (Pasquill "G" stability), the longest persistence of one wind direction is 9 hours (south-southwest). For calm conditions, the longest persistence of G stability is 8 hours (see Subsection 2.3.2.1.6).

Based on the regional airflow characteristics for the Braidwood site, it is concluded that the constant mean wind direction model which utilizes single-station meteorological data is an acceptable method of calculating χ/Q values for the Braidwood site.

2.3.5.2.1 Joint Frequency Distribution of Wind Direction, Wind Speed and Stability (For TID-14844 based dose analyses)

The effluents released from the Braidwood Station vent stack may be considered either elevated or ground-level releases, depending on the ratio of the vertical exit velocity of the stack discharge to the horizontal wind speed. To accommodate this variation and to utilize the appropriate meteorological data obtained at the 34-foot and 203-foot levels of the Braidwood onsite meteorological tower, a composite joint frequency distribution of wind speed, wind direction, and stability class for each of the elevated or ground-level release conditions is determined from an hour-by-hour scan of wind speed data recorded at both levels of the meteorological tower.

The stack height level wind speed is chosen as the most representative for determining the release condition. Therefore, wind speeds at the 203-foot level of the meteorological tower are used for determining the release condition at the 200-foot stack height level.

To determine if an elevated or ground-level release condition exists, a ratio, V_R, of the vertical exit velocity of the effluent plume to horizontal wind speed at the stack height level is defined:

$$V_R = W_o / u \tag{2.3-4}$$

where:

W_o = vertical exit velocity of the effluent plume, and

u = stack height wind speed.

The ratio V_R is then used in the following equations to define an entrainment coefficient, E_t (Reference 24):

$$E_t = 1.00 \text{ for } V_R < 1.0$$

$$E_t = 2.58 - 1.58\, V_R \text{ for } 1 \leq V_R \leq 1.5$$

$$E_t = 0.3 - 0.06\, V_R \text{ for } 1.5 < V_R < 5.0$$

$$E_t = 0 \text{ for } V_R \geq 5.0 \tag{2.3-5}$$

For the hour being scanned, the release is considered an elevated release $(1-E_t) \times 100\%$ of the time and a ground-level release E_t 100% of the time. The total time duration of the elevated release, t_e, is then calculated as:

$$t_e = (1-E_t) \times 1 \text{ hour} \tag{2.3-6}$$

and the total time duration of the ground level release, t_g is calculated as:

$$t_g = E_t \times 1 \text{ hour} \tag{2.3-7}$$

The elevated and ground level joint frequencies $(F_{isd})e$ and $(F_{isd})g$ for each stability class i, wind speed s, and wind direction d are then determined as follows:

$$(F_{isd})e = \frac{\sum_{j=1}^{T} (t_{isd})e,j}{T} \tag{2.3-8}$$

$$(F_{isd})g = \frac{\sum_{j=1}^{T} (t_{isd})g,j}{T} \tag{2.3-9}$$

where:

$(F_{isd})_e$ = joint frequency of stability class i, wind speed class s, and wind sector d applicable to the elevated release condition as derived from the 199-foot level meteorological data;

$(F_{isd})_g$ = joint frequency of stability class i, wind speed class s, and wind sector d, applicable to the ground-level release condition as derived from the 30-foot level meteorological data;

$(t_{isd})_{e,j}$ = time duration of wind speed class s, wind direction d, and stability class i at the 199-foot meteorological tower level during hour j;

$(t_{isd})_{g,j}$ = time duration of wind speed class s, wind direction d, and stability class i at the 30-foot meteorological tower level during hour j; and

T = total number of scanned hours with valid data.

It is noted that:

$$\sum_{i,s,d} (F_{isd})_e + \sum_{i,s,d} (F_{isd})_g = 1 \qquad (2.3\text{-}10)$$

2.3.5.2.2 Effective Release Height (For TID-14844 based dose analyses)

For an elevated release, the effective release height, h_e, is defined below:

$$h_e = h_s + h_{pr} - C \qquad (2.3\text{-}11)$$

where:

h_e = effective release height,

h_s = physical stack height,

h_{pr} = plume rise (defined below), and

C = correction for low exit velocity (defined below).

Because of the modest relief surrounding the Braidwood Station site, receptor terrain heights have not been considered in determining the effective release height (see Subsection 2.3.2.3). The plume rise, h_{pr}, is calculated from the following momentum plume rise equations (Reference 25):

For neutral or unstable conditions, the smaller value from the two following equations is used:

$$h_{pr} = 1.44 \left(\frac{W_o}{u}\right)^{\frac{2}{3}} \bullet \left(\frac{X}{D}\right)^{\frac{1}{3}} \bullet D \qquad (2.3\text{-}12)$$

or

$$h_{pr} = 3.0\left(\frac{W_o}{u}\right)D \qquad (2.3\text{-}13)$$

where:

h_{pr} = plume rise,

W_o = vertical exit velocity of effluent plume,

X = downwind distance from release point,

u = stack height wind speed, and

D = internal stack diameter.

For stable conditions, the results from Equation 2.3-12 and Equation 2.3-13 are compared with the results from the following two equations, and the smallest among the values obtained from Equations 2.3-12 through 2.3-15 are then used:

$$h_{pr} = 4\left(\frac{F_m}{S}\right)^{\frac{1}{4}} \qquad (2.3\text{-}14)$$

$$h_{pr} = 1.5\left(\frac{F_m}{u}\right)^{\frac{1}{3}} S^{-\frac{1}{6}} \qquad (2.3\text{-}15)$$

where:

h_{pr} = plume rise,

F_m = momentum flux parameter = $W_o^2 \; \frac{D}{2} \; 2$,

S = a stability parameter = $\frac{g}{T} \; \frac{\partial\theta}{\partial Z}$

g = acceleration of gravity,

T = ambient air temperature, and

$\frac{\partial\theta}{\partial Z}$ = vertical potential temperature gradient.

In the calculations S was defined as 8.7×10^{-4} sec^{-2} for E stability, 1.75×10^{-3} sec^{-2} for F stability, and 2.45×10^{-3} sec^{-2} for G stability.

When the vertical exit velocity is less than 1.5 times the horizontal wind speed, a correction, for stack downwash, C, is subtracted from the effective stack height as shown in equation 2.3-11 (Reference 24):

$$C = 3\,(1.5 - W_o / u)\,D \tag{2.3-16}$$

where:

C = downwash correction factor,

D = inside diameter of the stack,

u = wind speed at stack height, and

W_o = vertical exit velocity of the plume.

For ground-level releases, the effective release height at all times is zero ($h_e = 0$).

2.3.5.2.3 Annual Average Atmospheric Dilution Factor (For TID-14844 based dose analyses)

Using the joint frequency distributions developed in Subsection 2.3.5.2.1, the sector-averaged dispersion equations are used to calculate annual average dispersion factors for locations of interest. Equation 2.3-17, given below, is used to calculate the dispersion factor for elevated release conditions:

$$(\chi/Q)_{de} = \frac{1}{B}\sqrt{\frac{2}{\pi}}\sum_{i,s}\frac{(F_{isd})_e \exp\left[-\frac{h_e^2}{2\,\sigma_{zi}^2}\right]}{\sigma_{zi}\, u_s\, X} \tag{2.3-17}$$

where:

$(\chi/Q)_{de}$ = atmospheric dilution factor (sec/m^3) at a distance X in downwind sector of wind direction d for elevated release conditions;

B = sector width for 22.5° sector = 0.3927 radians;

$(F_{isd})_e$ = joint frequency of stability class i, wind speed class s, and wind sector d, applicable to the elevated release condition;

h_e = effective release height;

σ_{zi} = vertical standard deviation of contaminant concentration at distance X for stability class i (Reference 20);

X = downwind distance from release point (m); and

u_s = stack height wind speed class s.

For ground level release the following equation is used:

$$(\chi/Q)_{dg} = \frac{1}{B}\sqrt{\frac{2}{\pi}}\sum_{i,s}\frac{(F_{isd})_g}{\sigma^{*}_{zi}\ u_s\ X} \qquad (2.3\text{-}18)$$

where:

$(\chi/Q)_{dg}$ = atmospheric dilution factor (sec/m^3), at a distance X in downwind sector of wind direction d for ground-level release conditions;

B = sector width for 22.5° sector = 0.3297 radians;

$(F_{isd})_g$ = joint frequency of stability class i, wind speed class s, and wind sector d applicable to the ground-level release condition;

h_e = effective release height (m);

σ^{*}_{zi} = vertical standard deviation of contaminant concentration at a distance X for stability class i, corrected for additional dispersion within the reactor building cavity (Reference 24)

= $(\sigma^2_{zi} + CA)^{1/2} \leq \sigma_{zi}\ x\ \sqrt{3}$

where:

C = building shape factor = 0.5;

A = minimum cross-sectional area of containment building = 2700 m^2;

X = downwind distance from release point; and

u_s = 30-foot level wind speed class s.

The values of the $(\chi/Q)_{de}$ and $(\chi/Q)_{dg}$ calculated at each downwind distance are added together to give the total annual average dispersion factor, $(\chi/Q)_d$' at that distance:

$$(\chi / Q)_d = (\chi / Q)_{de} + (\chi / Q)_{dg} \tag{2.3-19}$$

Annual average χ/Q for the 200-foot Braidwood vent stack at the site boundary distances and at various distances out to 50 miles (80.5 km) are presented in Tables 2.3-39 and 2.3-40.

2.3.6 Short-term (Accident) Diffusion Estimates (Alternative Source Term χ/Q Analysis)

2.3.6.1 Objective

Estimates of atmospheric diffusion (χ/Q) at the Exclusion Area Boundary (EAB) and the outer boundary of the Low Population Zone (LPZ) are calculated for the regulated short-term (accident) time-averaging periods of 0-2 hrs, 2-8 hrs, 8-24 hrs, 1-4 days, and 4-30 days.

2.3.6.2 Meteorological Data

The Braidwood onsite meteorological tower database for the five-year period, 1994-1998, was applied in the modeling analyses. Wind speed and direction data taken at 34 ft and 203 ft and the vertical temperature difference data measured between 199 ft and 30 ft were utilized. “Calm” wind speeds were assigned a value of 0.4 mph (1/2 the instrument threshold starting speed value). The combined data recovery of wind speed, wind direction, and stability data exceeded the RG 1.23 (Reference 28) goal of 90 percent for each of the 5 years (1994 through 1998).

2.3.6.3 Calculation of χ/Q at the EAB and LPZ

The calculation of χ/Q at the EAB (i.e., 485 m) for postulated releases from the Unit 1 and Unit 2 outer Containment Wall, and at the LPZ (1810 m) for a postulated release from the midpoint between the two reactors, utilized the NRC-recommended model PAVAN (Reference 26), which implements Regulatory Guide 1.145 methodology. These releases do not qualify as “elevated releases” as defined by Regulatory Guide 1.145 (Reference 27); therefore, they are executed as “ground” type releases.

The calculation of χ/Q at the EAB and LPZ by PAVAN in accordance with Regulatory Guide 1.145 for ground-level releases is based on the following equations:

$$\chi/Q = \frac{1}{\overline{U}_{10}\left(\pi\sigma_y\sigma_z + A/2\right)} \tag{2.3.6-1}$$

$$\chi/Q = \frac{1}{\overline{U}_{10}\left(3\pi\sigma_y\sigma_z\right)} \tag{2.3.6-2}$$

$$\chi/Q = \frac{1}{\overline{U}_{10}\pi\Sigma_y\sigma_z} \qquad (2.3.6\text{-}3)$$

where:

χ/Q is relative concentration, in sec/m^3.

π is 3.14159.

$\overline{U}_{10}$ is wind speed at 10 meters above plant grade, in m/sec.

σ_y is lateral plume spread, in meters, a function of atmospheric stability and distance.

σ_z is vertical plume spread, in meters, a function of atmospheric stability and distance.

Σ_y is lateral plume spread, in meters, with meander and building wake effects (in meters), a function of atmospheric stability, wind speed, and distance [for distances of 800 m or less, $\Sigma_y = M\sigma_y$, where M is determined from Reg. Guide 1.145 Fig. 3; for distances greater than 800 m, $\Sigma_y = (M-1)\sigma_{y\,800\,m} + \sigma_y$.

A is the smallest vertical-plane cross-sectional area of the reactor building, in m^2. (Other structures or a directional consideration may be justified when appropriate.)

Plume meander is only considered during neutral (D) or stable (E, F, or G) atmospheric stability conditions. For such, the higher of the values resulting from Equations 2.3.6-1 and 2.3.6-2 is compared to the value of Equation 2.3.6-3 for meander, and the lower value is selected. For all other conditions (stability classes A, B, or C), meander is not considered and the higher χ/Q value of Equations 2.3.6-1 and 2.3.6-2 is selected.

The χ/Q values calculated at the EAB and LPZ based on meteorological data values representing a 1-hour average are assumed to apply for an entire 2-hour period.

To determine the “maximum sector 0-2 hour χ/Q” value at the EAB, PAVAN constructs a cumulative frequency probability distribution (probabilities of a given χ/Q value being exceeded in that sector during the total time) for each of the 16 sectors using the χ/Q values calculated for each hour of data. This probability is then plotted versus the χ/Q values and a smooth curve is fitted to form an upper bound of the computed points. For each of the 16 curves, the χ/Q value that is exceeded 0.5 percent of the total hours is selected and designated as the sector χ/Q value. The highest of the 16 sector χ/Q values is the maximum sector χ/Q.

The “maximum sector 0-2 hour χ/Q” value at the LPZ is calculated analogously to the EAB. Determination by PAVAN of the LPZ maximum sector χ/Q for periods greater than 0-2 hours is based on a logarithmic interpolation between the 2-hour sector χ/Q and the annual average χ/Q for the same sector. For each time period, the highest of these 16 sector χ/Q values is identified as the maximum sector χ/Q value. The maximum sector χ/Q values will, in most cases, occur in the same sector. If they do not occur in the same sector, all 16 sets of values are used in dose assessment requiring time-integrated concentration considerations. The set that results in the highest time-integrated dose within a sector is considered the maximum sector χ/Q.

The “5% overall site χ/Q” values for the EAB and LPZ are each determined by constructing an overall cumulative probability distribution for all directions. The 0-2 hour χ/Q values computed by PAVAN are plotted versus their probability of being exceeded, and an upper bound curve is fitted by the model. From this curve, the 2-hour χ/Q value that is exceeded 5% of the time is determined. PAVAN then calculates the 5% overall site χ/Q at the LPZ for intermediate time periods by logarithmic interpolation of the maximum of the 16 annual average χ/Q values and the 5% 0-2 hour χ/Q values.

2.3.6.3.1 PAVAN Meteorological Database

The meteorological database utilized for the EAB and LPZ χ/Q calculations were prepared for use in PAVAN by transforming the five years (i.e. 1994-1998) of hourly meteorological tower data observations into a joint wind speed-wind direction-stability class occurrence frequency distribution as shown in Tables 2.3-50 and 2.3-51. In accordance with Regulatory Guide 1.145, atmospheric stability class was determined by vertical temperature difference between the 199 ft and the 30 ft level, and wind direction was distributed into 16- 22.5° sectors.

Seven (7) wind speed categories were defined according to Regulatory Guide 1.23 (Reference 28) with the first category identified as "calm". In the equations shown in Section 2.3.6.3, it should be noted that wind speed appears as a factor in the denominator. This presents an obvious difficulty in making calculations for hours of calm. The minimum wind speed (i.e. based on the wind instrument starting threshold) was set to 0.80 mph, and “calm” winds were assigned a value of 0.4 mph (1/2 the threshold value). The procedure used by PAVAN assigns a direction to each calm hour according to the directional distribution for the lowest non-calm wind-speed class. This procedure is performed separately for the calms in each stability class.

A midpoint was also assumed between each of the Regulatory Guide 1.23 wind speed categories, Nos. 2-6, as to be inclusive of all wind speeds. The wind speed categories have therefore been defined as follows:

CATEGORY NO.	REGULATORY GUIDE 1.23, REV. 0 SPEED INTERVAL (MPH)	PAVAN-ASSUMED SPEED INTERVAL (MPH)
1 (Calm)	0 to < 1	0 to <0.80
2	1 to 3	≥0.80 to <3.5
3	4 to 7	≥3.5 to <7.5
4	8 to 12	≥7.5 to <12.5
5	13 to 18	≥12.5 to <18.5
6	19 to 24	≥18.5 to <24
7	>24	≥24

Based on a commitment made to the NRC, finer wind speed categories provided in the latest appropriate regulatory guidance are to be used the next time the dose consequence calculations associated with the LOCA, MSLB, CREA, LRA, SGTR, and FHA events are revised. Finer wind speed categories from RG 1.23 Rev. 1 (Reference 33) are used for the revised analysis. The finer wind speed categories are defined as follows:

CATEGORY NO.	REGULATORY GUIDE 1.23 REV. 1 SPEED INTERVAL (M/S)	PAVAN-ASSUMED SPEED INTERVAL(M/S)	PAVAN-ASSUMED SPEED INTERVAL (MPH)
1 (Calm)	<0.5	<0.36	<0.80
2	0.5 to 1.0	≥0.36 to <1.05	≥0.80 to <2.35
3	1.1 to 1.5	≥1.05 to <1.55	≥2.35 to <3.47
4	1.6 to 2.0	≥1.55 to <2.05	≥3.47 to <4.56
5	2.1 to 3.0	≥2.05 to <3.05	≥4.56 to <6.82
6	3.1 to 4.0	≥3.05 to <4.05	≥6.82 to <9.06
7	4.1 to 5.0	≥4.05 to <5.05	≥9.06 to <11.30
8	5.1 to 6.0	≥5.05 to <6.05	≥11.30 to <13.53
9	6.1 to 8.0	≥6.05 to <8.05	≥13.53 to <18.01
10	8.1 to 10.0	≥8.05 to <10.0	≥18.01 to <22.40
11	>10.0	≥10.0	≥22.40

2.3.6.3.2 PAVAN Model Input Parameters

Both the Unit 1 and Unit 2 Containment Building outer wall and the midpoint between the two reactors do not qualify as “elevated” release locations per Regulatory Guide 1.145; therefore, PAVAN requires that release height be assigned a value of 10 m. For this non-elevated release, EAB and LPZ receptor terrain elevation is assumed to be equal to plant grade.

The Containment Building height of 60.7 m above grade and the calculated Containment Building vertical cross-sectional area of 2,916.7 m^2 were used for both the EAB and LPZ PAVAN computations (see Braidwood Drawing A-4).

2.3.6.3.3 PAVAN EAB and LPZ χ/Q

Atmospheric χ/Q diffusion estimates predicted by PAVAN at the EAB and LPZ are summarized below.

RG 1.23 REV. 0
EAB AND LPZ χ/Q SUMMARY (sec/m^3)
BRAIDWOOD STATION

Release Point	Receptor	0-2 hour	0-8 hour	8-24 hour	1-4 day	4-30 day
Unit 1 and Unit 2 Outer Containment Wall	EAB (485 m)	4.78E-04	2.48E-04	1.79E-04	8.77E-05	3.16E-05
Midpoint between the Unit 1 and Unit 2 Reactors	LPZ (1810 m)	9.32E-05	4.50E-05	3.12E-05	1.41E-05	4.54E-06

RG 1.23 REV 1
EAB and LPZ χ/Q SUMMARY (sec/m^3)
BRAIDWOOD STATION

Release Point	Receptor	0-2 hour	0-8 hour	8-24 hour	1-4 day	4-30 day
Unit 1 and Unit 2 Outer Containment Wall	EAB (485 m)	5.71E-04	2.86E-04	2.03E-04	9.60E-05	3.28E-05
Midpoint between the Unit 1 and Unit 2 Reactors	LPZ (1810 m)	1.10E-04	5.13E-05	3.51E-05	1.53E-05	4.68E-06

2.3.6.4 Calculation of χ/Q at the Control Room Intakes

Calculations of atmospheric diffusion (χ/Q) are made for each of the two Control Room Intakes (i.e. Fresh Air and Turbine Building Emergency Air) resulting from releases from the following four points: The Containment Wall, the Plant Vent, the PORVs/Safety Valves, and the Main Steam Line Break (MSLB) for periods of 2, 8, and 16 hours, and for 3 and 26 days. The NRC-sponsored computer code ARCON96 (Reference 29), is utilized in accordance with the procedures in Regulatory Guide (RG) 1.194 (Reference 30).

2.3.6.4.1 ARCON96 Model Analysis

The four releases do not qualify as elevated per RG 1.194, since none are equal to or greater than 2.5 times the height of the adjacent structures; therefore, ARCON96 is executed in ground release mode. The basic model for a ground-level release is as follows:

$$\frac{\chi}{Q} = \frac{1}{\pi\sigma_y\sigma_z U}\exp\left[-0.5\left(\frac{y}{\sigma_y}\right)^2\right] \qquad (2.3.6\text{-}4)$$

where:

χ/Q = relative concentration (concentration divided by release rate)[(ci/m^3)/(ci/s)]

σ_y, σ_z = lateral and vertical diffusion coefficients (m)

U = wind speed (m/s)

y = lateral distance from the horizontal centerline of the plume (m)

This equation assumes that the release is continuous, constant, and of sufficient duration to establish a representative mean concentration. It also assumes that the material being released is reflected by the ground. Diffusion coefficients are typically determined from atmospheric stability and distance from the release point using empirical relationships. A diffusion coefficient parameterization from the NRC PAVAN and XOQDOQ (Reference 31) codes is used for σ_y and σ_z.

The diffusion coefficients have the following general form:

$$\sigma = a\ x^{b} + c$$

were x is the distance from the release point, in meters; and a , b, and c are parameters that are functions of stability class. These parameters are defined for three (3) downwind distance ranges: 0 to 100 m, 100 to 1000 m, and greater than 1000 m. The diffusion coefficient parameter values may be found in the listing of Subroutine NSIGMA1 in Appendix A of NUREG/CR-6331 Rev. 1.

Diffusion coefficient adjustments for wakes and low wind speeds are also incorporated by ARCON96 as follows:

In order to estimate diffusion in building wakes, composite wake diffusion coefficients, Σ_y and Σ_z, replace σ_y and σ_z. The composite wake diffusion coefficients are defined as follows:

$$\Sigma_y = \left(\sigma_y{}^2 + \Delta\sigma_{y1}{}^2 + \Delta\sigma_{y2}{}^2\right)^{1/2} \qquad (2.3.6\text{-}5)$$

$$\Sigma_z = \left(\sigma_z{}^2 + \Delta\sigma_{z1}{}^2 + \Delta\sigma_{z2}{}^2\right)^{1/2} \qquad (2.3.6\text{-}6)$$

The variables σ_y and σ_z are the general diffusion coefficients, $\Delta\sigma_{y1}$ and $\Delta\sigma_{z1}$ are the low wind speed corrections, and $\Delta\sigma_{y2}$ and $\Delta\sigma_{z2}$ are the building wake corrections. These corrections are described and evaluated in Ramsdell and Fosmire (Reference 32). The low wind speed corrections are:

$$\Delta\sigma_{y1}{}^2 = 9.13\times10^5\left[1-\left(1+\frac{x}{1000U}\right)\exp\left(\frac{-x}{1000U}\right)\right] \qquad (2.3.6\text{-}7)$$

$$\Delta\sigma_{z1}{}^2 = 6.67\times10^2\left[1-\left(1+\frac{x}{100U}\right)\exp\left(\frac{-x}{100U}\right)\right] \qquad (2.3.6\text{-}8)$$

The variable x is the distance from the release point to the receptor, in meters, and U is the wind speed in meters per second. It is appropriate to use the slant range distance for x because these corrections are made only when the release is assumed to be at the ground level and the receptor is assumed to be on the axis of the plume. The diffusion coefficient corrections that account for enhanced diffusion in the wake have a similar form. These corrections are:

$$\Delta\sigma_{y2}{}^2 = 5.24\times10^{-2}U^2A\left[1-\left(1+\frac{x}{10\sqrt{A}}\right)\exp\left(\frac{-x}{10\sqrt{A}}\right)\right] \qquad (2.3.6\text{-}9)$$

$$\Delta\sigma_{z2}{}^2 = 1.17\times10^{-2}U^2A\left[1-\left(1+\frac{x}{10\sqrt{A}}\right)\exp\left(\frac{-x}{10\sqrt{A}}\right)\right] \qquad (2.3.6\text{-}10)$$

The constant A is the cross-sectional area of the building.

A conservative upper limit placed on Σ_y is the standard deviation associated with a concentration uniformly distributed across a sector with width equal to the circumference of a circle, and with a radius equal to the distance between the source and receptor. This value is

$$\Sigma_{ymax} = \frac{2\pi x}{\sqrt{12}} \approx 1.81x \qquad (2.3.6\text{-}11)$$

2.3.6.4.1.1 ARCON96 Meteorological Database

The 1994-1998 meteorological database utilized by ARCON96 consists of hourly meteorological data observations of wind speed and direction measured at 34 and 203 ft, and delta temperature stability class measured between 199 and 30 ft.

The calm wind occurrences, defined to have a value of 0.4 mph (1/2 the wind instrument threshold starting speed), were reset to the ARCON96 default value minimum wind speed value of 0.5 meters per second per RG 1.194, Table A-2.

2.3.6.4.1.2 ARCON96 Input Parameters

ARCON96 is executed for each source/receptor combination shown below:

1) Unit 1 Containment Wall to Control Room Fresh Air Intake
2) Unit 1 Containment Wall to Control Room Turbine Building Emergency Air Intake
3) Unit 1 Plant Vent to Control Room Fresh Air Intake
4) Unit 1 Plant Vent to Control Room Turbine Building Emergency Air Intake
5) Unit 1 PORVs/Safety Valves to Control Room Fresh Air Intake
6) Unit 1 PORVs/Safety Valves to Control Room Turbine Building Emergency Air Intake
7) Unit 1 MSLB to Control Room Fresh Air Intake
8) Unit 1 MSLB to Control Room Turbine Building Emergency Air Intake
9) Unit 2 Containment Wall to Control Room Fresh Air Intake
10) Unit 2 Containment Wall to Control Room Turbine Building Emergency Air Intake
11) Unit 2 Plant Vent to Control Room Fresh Air Intake
12) Unit 2 Plant Vent to Control Room Turbine Building Emergency Air Intake
13) Unit 2 PORVs/Safety Valves to Control Room Fresh Air Intake
14) Unit 2 PORVs/Safety Valves to Control Room Turbine Building Emergency Air Intake
15) Unit 2 MSLB to Control Room Fresh Air Intake
16) Unit 2 MSLB to Control Room Turbine Building Emergency Air Intake

All release scenarios are conservatively assumed to have a zero (0) vertical velocity, exhaust flow and stack radius. Other ARCON96 input parameter values were set in accordance with RG 1.194, Table A-2 (e.g. surface roughness length = 0.2 m; wind direction window = 90 degrees, 45 degrees on either side of line of sight from source to receptor; minimum wind speed = 0.5 m/s; and averaging sector width constant = 4.3).

Each Containment Wall scenario is modeled as a "diffuse area" source in ARCON96. The method of modeling this release as a diffuse area source is in conformance with RG 1.194, as described in Appendix A. All other scenarios are modeled as ground-level point sources.

The area source representation in ARCON96 requires the building cross-sectional area to be calculated from the maximum building dimensions projected onto a vertical plane perpendicular to the line of sight from the building to the intake. The Containment Building, with a height of 195 ft (not including the dome portion above the collar) and a width of 161 ft has an area of 31,395 ft^2 (2,916.7 m^2) (see Braidwood Drawing A-4). The diffuse area source also requires the release height to be assumed at the vertical center of the projected area, and the initial diffusion coefficients to be specified. Per RG 1.194, Section 3.2.4.4, the initial diffusion coefficients are calculated as follows:

$$\sigma_{Y_0} = \frac{Width_{area\ source}}{6} \quad (2.3.6\text{-}12) \qquad \sigma_{Z_0} = \frac{Height_{area\ source}}{6} \quad (2.3.6\text{-}13)$$

$$\sigma_{Y_0} = \frac{161 ft}{6} = 26.83\ ft = 8.18\ m \qquad \sigma_{Z_0} = \frac{195 ft}{6} = 32.5\ ft = 9.9\ m$$

The remaining three releases at each station (i.e. Plant Vent, PORVs/Safety Valves, and MSLB) are each modeled as a point source. Per RG 1.194 Table A-2, the building area perpendicular to the wind direction should be utilized. For the PORVs/Safety Valves, the Containment Building area of 2,916.7 m^2 was utilized for both stations. There is no change in this building area with a change in wind direction due to its circular shape. The Auxiliary Building area was utilized for the Plant Vent and MSLB scenarios.

ARCON96 requires input of a horizontal source-receptor distance, defined in RG 1.194 Section 3.4 as "the shortest horizontal distance between the release point and the intake". However, for releases in building complexes, a "taut string length" can be utilized as justifiable. For the MSLB to Control Room Fresh Air Intake scenarios, this "taut string length" was utilized to account for the intervening Auxiliary Building. As per NRC interpretation of this RG, when the "taut string length" is utilized, the intake and release heights should be set equal to each other so as not to effectively double-count the advantage of the slant distance that ARCON96 calculates. Therefore, the intake height was set equal to the release height of 7.9 m.

The ARCON96 input parameter data used in calculating the χ/Q values are summarized in Table 2.3-52.

2.3.6.4.1.3 ARCON96 Control Room Intake χ/Q

A summary of the atmospheric diffusion estimates for the two Control Room Intakes is shown in Table 2.3-53.

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TABLE 2.3-1

CLIMATOLOGICAL DATA FROM WEATHER STATIONS
SURROUNDING THE BRAIDWOOD SITE*

PARAMETER	STATION PEORIA	 CHICAGO (MIDWAY)	 ARGONNE
Temperature (°F)			
Annual average	50.8	50.6	47.7
Maximum	102 (July 1966)	104 (June 1953)	101 (July 1956)
Minimum	-20 (January 1963)	-16 (January 1966)	-20 (January 1963)
Degree-days	6098	6127	6911
Relative Humidity (%)			
Annual average at:			
6 a.m.	83	76	87
12 noon	62	59	62
Wind			
Annual average speed (mph)	10.3	10.4	7.6**
Prevailing direction	S	W	SW
Fastest mile:			
Speed (mph)	75 (July 1953)	60 (November 1952)	64*** (July 1957)
Direction	NW	SW	+
Precipitation (inches)			
Annual average	35.06	34.44	31.49
Monthly maximum	13.09 (September 1961)	14.71 (September 1961)	13.17 (September 1961)
Monthly minimum	0.03 (October 1964)	0.02 (October 1964)	0.03 (January 1961)
24-hour maximum	5.06 (April 1950)	6.24 (July 1957)	6.54 (+)

TABLE 2.3-1 (Cont'd)

PARAMETER	STATION PEORIA	 CHICAGO (MIDWAY)	 ARGONNE
Snowfall (inches)			
Annual average	23.4	40.4	+
Monthly maximum	18.9 (December 1973)	33.3 (December 1951)	+
Monthly minimum	10.2 (December 1973)	19.8 (January 1967)	+
Mean Annual (Number of Days)			
Precipitation > 0.1 inch	111	123	110
Snow, sleet, hail > 1.0 inch	8	12	+
Thunderstorms	49	40	+
Heavy fog	21	13	+
Maximum temperature > 90°F	17	21	+
Minimum temperature > 32°F	132	119	+

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* The data presented in this table are based upon References 2, 3, and 4. For the Peoria Midway data, the periods of record used for these statistics range from 7 to 37 years in length within the time period 1940 to 1976. The period for the Argonne data is the 15-year period 1950 to 1964.

** Wind at 19-foot tower level.

*** Peak gust wind at 19-foot level.

\+ Data are not recorded.

Table 2.3-2

MEASURES OF GLAZING IN VARIOUS SEVERE WINTER STORMS FOR THE STATE OF ILLINOIS*

STORM DATE	RADIAL THICKNESS OF ICE ON WIRE (in.)	RATIO OF ICE WEIGHT TO WEIGHT OF 0.25-in. TWIG	WEIGHT OF ICE (oz) ON 1 FOOT OF STANDARD (#12) WIRE	CITY	STATE SECTION
2-4 February 1883	-	-	11	Springfield	WSW
20 March 1912	0.5	-	-	Decatur	C
21 February 1913	2.0	-	-	La Salle	NE
11-12 March 1923	1.6	-	12	Marengo	NE
17-19 December 1924	1.2	15:1	8	Springfield	WSW
22-23 January 1927	1.1	-	2	Cairo	SE
31 March 1929	0.5	-	-	Moline	NW
7-8 January 1930	1.2	-	-	Carlinville	WSW
1-2 March 1932	0.5	-	-	Galena	NW
7-8 January 1937	1.5	-	-	Quincy	W
31 December 1947 - 1 January 1948	1.0	-	72	Chicago	NE
10 January 1949	0.8	-	-	Macomb	W
8 December 1956	0.5	-	-	Alton	WSW
20-22 January 1959	0.7	12:1	-	Urbana	E
26-27 January 1967	1.7	17:1	40	Urbana	E

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*Based on Reference 15.

TABLE 2.3-3

WIND-GLAZE THICKNESS RELATIONS FOR FIVE PERIODS OF GREATEST SPEED AND GREATEST THICKNESS*

	FIVE PERIODS WHEN FIVE FASTEST 5-MINUTE SPEEDS WERE REGISTERED		FIVE PERIODS WHEN FIVE GREATEST ICE THICKNESSES WERE MEASURED	
RANK	SPEED (mph)	ICE THICKNESS (in.)	ICE THICKNESS (in.)	SPEED (mph)
1	50	0.19	2.87	30
2	46	0.79	1.71	18
3	45	0.26	1.50	21
4	40	0.30	1.10	28
5	35	0.78	1.00	18

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*From data collected throughout the United States during the period 1926-1937. Based on Reference 15.

TABLE 2.3-4

ANNUAL WIND ROSE DATA FOR THE 30-FOOT LEVELS AT THE BRAIDWOOD SITE (1974-1976)*

30-FOOT LEVEL

Speed (mps)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL
Calm																	2.25
0.26 - 1.50	.47	.51	.62	1.28	1.45	1.14	.87	.70	.60	.63	.54	.53	.61	.65	.74	.51	11.84
1.51 - 3.00	1.23	1.09	1.26	1.73	1.69	1.50	2.00	1.74	1.86	1.97	1.80	1.83	1.54	1.59	1.38	1.41	25.64
3.01 - 7.00	2.86	2.64	2.60	1.33	.95	1.18	2.46	3.27	5.54	5.80	4.12	3.14	3.01	3.62	3.63	3.17	49.32
GT. 7.00	.36	.47	.24	.03	.02	.05	.18	.71	2.32	2.26	1.01	.63	.73	.87	.54	.53	10.95
Totals	4.94	4.72	4.74	4.38	4.13	3.88	5.54	6.43	10.34	10.66	7.47	6.14	5.91	6.75	6.32	5.63	100.00

199-FOOT LEVEL

Speed (mps)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL
Calm																	.25
0.26 - 1.50	.11	.13	.14	.10	.13	.09	.09	.10	.10	.12	.12	.11	.15	.11	.17	.12	1.90
1.51 - 3.00	.54	.49	.48	.54	.51	.34	.41	.41	.39	.42	.40	.40	.49	.53	.62	.55	7.53
3.01 - 7.00	3.12	2.65	2.75	3.12	3.06	2.35	2.76	2.52	3.22	3.64	3.39	3.21	3.70	3.69	3.62	3.72	50.50
GT. 7.00	1.24	1.21	1.40	.58	.66	1.31	1.76	2.17	4.66	6.09	4.42	2.91	2.76	3.27	3.22	2.16	39.81
Totals	5.00	4.50	4.78	4.34	4.37	4.10	5.02	5.21	8.37	10.27	8.32	6.63	7.11	7.59	7.63	6.55	100.00

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*Values in % of total observations

TABLE 2.3-5

PERSISTENCE OF WIND DIRECTION AT THE 30-FOOT LEVEL OF THE BRAIDWOOD SITE (1974-1976)*

PERSISTENCE (hr)	WIND DIRECTION																
	CALM	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW
1-3	157	503	481	476	466	493	478	587	677	770	827	785	998	743	702	648	570
4-6	22	56	60	59	57	47	33	62	65	137	123	88	71	60	82	87	86
7-9	10	16	16	9	10	6	10	15	20	52	46	24	19	13	17	12	18
10-12	2	3	5	4	3	2	1	3	4	11	17	4	4	0	5	7	4
13-15	1	1	2	3	0	0	0	2	0	7	5	1	1	1	2	3	3
16-18	0	0	0	0	0	0	0	2	0	2	3	0	0	1	1	1	1
19-21	0	0	0	2	0	0	0	0	1	2	0	0	0	1	1	0	0
22-24	0	0	0	0	0	0	0	0	0	2	0	0	0	0	1	0	1
25-27	0	0	0	0	0	0	0	0	0	1	1	0	0	0	0	0	0
28-30	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
31-33	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
34-39	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
40-45	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
>45	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

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*Values tabulated in number of occurrences.

TABLE 2.3-6

PERSISTENCE OF WIND DIRECTION AT THE 199-FOOT LEVEL OF THE BRAIDWOOD SITE (1974-1976)*

PERSISTENCE (hr)	WIND DIRECTION																
	CALM	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW
1-3	32	466	462	444	415	423	420	437	511	627	714	739	668	701	732	672	563
4-6	1	58	48	52	60	50	47	53	67	110	148	93	77	94	84	82	82
7-9	0	24	12	16	16	19	14	25	19	31	39	30	18	22	19	21	30
10-12	0	5	5	7	6	4	4	6	3	7	21	13	6	8	12	16	10
13-15	0	3	1	4	0	0	1	8	4	7	4	6	2	0	1	3	5
16-18	0	0	2	0	0	0	0	0	0	5	6	1	1	0	0	1	1
19-21	0	1	1	0	0	0	0	0	0	1	2	0	0	0	1	1	0
22-24	0	0	0	0	1	0	0	0	0	2	0	0	0	0	0	1	0
25-27	0	0	0	0	0	0	0	0	0	0	0	1	0	0	0	0	0
28-30	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
31-33	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0	0
34-39	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1
45-45	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
>45	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

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*Values tabulated in number of occurrences.

TABLE 2.3-7

PERSISTENCE AND FREQUENCY OF WIND DIRECTION AT PEORIA (1966-1975)*

PERSISTENCE	WIND DIRECTION																
(hr)	CALM	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW
3	469	747	578	607	620	680	596	770	1026	1132	947	894	829	845	931	834	694
6	142	230	91	147	192	195	111	200	264	445	157	161	166	268	257	229	152
9	49	95	28	51	73	59	44	59	93	249	48	40	47	99	93	84	45
12	29	37	10	17	25	32	7	23	34	134	17	7	9	51	48	21	14
15	4	21	3	7	5	12	2	8	9	84	3	2	5	24	21	10	11
18	0	6	1	0	6	3	2	3	1	48	1	1	0	13	13	2	1
21	0	6	0	0	4	5	0	2	2	37	0	2	0	4	8	4	0
24	0	2	0	2	1	2	0	0	0	21	0	0	1	3	2	0	0
27	0	2	0	0	0	0	0	0	0	20	0	1	0	2	0	1	0
30	0	0	0	0	0	1	0	0	0	9	0	0	0	3	2	1	0
33	0	0	0	0	0	0	1	0	0	5	0	0	0	1	0	0	0
36 - 39	0	0	0	0	0	0	0	0	0	12	0	0	0	0	3	0	0
42 - 45	0	0	0	0	0	1	0	0	0	5	0	0	0	0	0	0	0
45	0	0	0	0	0	0	0	0	0	4	0	0	0	0	0	0	0
TOTAL HOURS (In %)	3.55	6.4	3.1	4.0	4.9	5.3	3.4	5.2	7.0	17.3	5.1	4.8	4.7	7.5	7.6	6.0	4.3

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* Values tabulated in number of occurrences. Number of occurrences are based on observations made once every 3 hours, and each observation is assumed to persist for 3 hours.

TABLE 2.3-8

PERSISTENCE OF WIND DIRECTION FOR THE 19 - AND 150 - FOOT LEVELS AT ARGONNE (1950-1964)

(Number of Occurrences)

19 - FOOT LEVEL

HOURS OF PERSISTENCE			DIRECTION (36 POINTS)																																		
		CALM	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	MISSING
19-FT	1	924	1961	1868	1772	1829	1913	1955	1868	1789	1900	1929	1965	1989	1970	1929	1989	2160	2319	2308	2413	2402	2359	2519	2666	2810	2577	2626	2448	2440	2441	2331	2240	2180	2069	1941	2078	2140	438
LEVEL	2	310	814	720	676	756	781	762	731	713	762	822	790	782	767	802	872	904	916	950	901	920	1075	1094	1166	1182	1097	1058	1028	1019	973	906	913	889	864	851	828	833	93
	3	130	419	388	397	425	467	409	389	401	408	431	438	438	430	435	461	496	506	499	522	483	543	565	651	688	630	628	613	599	538	494	487	481	501	489	411	409	43
	4	51	261	248	238	252	303	248	251	279	260	240	251	258	246	278	308	314	309	313	316	338	405	440	463	460	435	388	391	346	353	317	343	333	324	281	244	276	20
	5	24	216	192	178	190	213	200	182	178	202	107	178	160	203	163	181	208	222	227	252	290	281	311	331	337	310	296	250	251	256	219	255	215	188	194	185	166	7
	6	26	162	139	133	154	157	138	134	146	152	148	119	107	117	146	146	161	181	184	201	191	229	244	250	241	238	206	204	186	182	175	158	167	169	130	133	141	5
	7	13	111	100	116	135	118	109	105	100	95	86	92	71	86	90	94	108	115	131	149	155	144	169	196	202	173	159	158	158	132	130	125	131	110	103	92	94	0
	8	9	100	99	85	98	89	96	95	94	81	69	56	44	59	75	69	92	85	123	128	125	143	147	165	123	141	139	123	114	124	106	118	117	101	87	85	92	0
	9	4	89	75	87	79	87	68	65	68	58	52	29	37	47	59	59	53	72	72	95	92	128	138	119	122	114	107	73	89	75	76	76	66	75	70	67	69	2
	10	1	70	73	73	58	65	64	44	47	36	29	29	31	43	32	46	47	66	68	86	81	100	112	80	98	103	81	75	96	70	69	83	67	62	65	70	51	0
	11	1	49	58	59	49	54	48	45	38	20	34	23	24	26	41	36	43	60	42	78	85	83	91	91	67	61	64	63	65	67	62	52	47	48	39	51	45	1
	12	2	43	56	45	48	38	33	39	39	25	15	16	15	24	21	25	24	36	50	50	68	75	71	68	54	66	62	47	66	43	46	42	40	46	31	38	49	0
	13	0	33	46	48	34	34	28	27	26	17	14	14	10	17	14	21	24	20	35	44	73	52	66	43	37	44	45	51	48	30	46	35	36	34	25	25	25	2
	14	0	20	29	26	36	38	34	21	22	7	8	16	8	14	17	13	24	23	39	51	46	63	38	46	42	32	40	34	36	39	34	40	32	24	19	20	21	2
	15	0	17	25	29	25	24	20	25	15	10	7	7	8	6	8	9	18	19	17	35	40	51	53	38	32	41	31	28	37	32	31	29	29	25	24	31	18	1
	16	0	22	20	21	21	22	12	12	10	3	13	12	9	6	8	5	15	21	25	39	46	46	35	22	25	31	26	29	27	29	22	25	23	16	27	20	18	0
	17	0	16	22	30	21	24	13	12	9	8	4	3	10	8	7	5	10	14	17	29	33	28	37	39	24	26	21	16	23	23	18	22	17	18	18	16	13	0
	18	0	22	29	16	14	19	12	10	3	3	6	9	3	2	5	4	7	12	24	24	32	29	33	27	20	21	24	26	20	30	19	15	14	10	18	18	9	0
	19	0	16	16	18	21	12	12	5	11	5	4	4	2	4	1	6	4	8	10	16	34	23	30	20	23	20	15	17	17	19	12	8	6	9	7	16	11	0
	20	0	1	14	13	9	7	10	11	7	3	3	3	11	1	3	5	3	8	10	21	26	21	21	27	6	13	13	11	17	19	8	13	11	6	12	10	13	1
	21-25	0	36	40	45	40	29	18	28	18	15	10	9	12	17	11	6	12	29	41	62	72	79	71	62	41	41	54	57	56	53	35	28	34	20	23	14	20	1
	26-30	0	16	20	13	20	13	13	11	11	7	4	3	1	2	6	3	3	8	14	36	27	42	30	32	23	23	28	23	28	26	18	18	20	17	28	24	9	1
	31-35	0	7	6	11	15	11	4	5	5	2	2	1	1	2	1	1	1	4	4	20	39	27	16	17	11	18	12	13	7	9	9	10	7	6	6	5	5	1
	36-40	0	6	5	4	5	4	1	5	6	1	0	0	1	0	0	0	0	1	4	7	13	10	20	7	2	9	13	5	11	4	2	6	5	1	5	3	4	0
	41-45	0	2	3	4	5	6	4	3	2	1	0	0	0	1	1	0	0	0	4	9	13	11	10	5	3	4	5	8	3	7	3	4	4	5	3	1	2	3
	>46	0	3	1	3	6	4	2	5	1	1	0	0	0	0	0	0	0	0	9	11	9	8	7	4	4	3	6	7	2	2	3	3	1	1	1	2	1	3
MAXIMUM PERSISTENCE		12	89	50	63	64	62	55	61	64	59	35	31	36	45	45	33	31	40	52	65	77	75	92	95	67	60	69	77	68	58	53	75	92	91	56	79	53	370

150 - FOOT LEVEL

HOURS OF PERSISTENCE			DIRECTION (36 POINTS)																																		
		CALM	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	MISSING
150-FT	1	413	1650	1577	1496	1512	1596	1600	1528	1441	1414	1522	1523	2545	1466	1485	1577	1668	1857	1851	1859	1858	1955	1981	2116	2254	2159	2150	2114	2125	2094	2039	1953	1833	1780	1713	1708	1756	338
LEVEL	2	81	722	654	609	638	699	679	639	590	647	641	641	676	702	680	734	764	808	820	810	828	909	959	954	1044	1038	1034	949	925	969	854	819	789	742	753	717	751	48
	3	37	376	370	348	362	419	427	401	374	368	376	411	433	408	386	437	458	484	458	480	467	483	550	634	660	616	581	590	558	510	510	490	466	447	399	391	391	20
	4	10	252	267	250	236	261	248	241	259	286	281	247	262	282	277	293	296	340	338	337	360	419	390	418	428	399	401	350	324	324	338	306	302	302	268	266	248	12
	5	1	183	191	174	156	170	196	191	166	189	202	199	168	179	192	199	214	208	235	253	274	266	312	324	315	326	258	246	239	231	233	238	214	211	210	175	152	8
	6	4	151	132	127	135	144	142	136	149	132	135	135	134	145	164	152	165	187	204	195	183	222	251	229	263	228	218	224	208	191	185	149	184	166	142	114	97	12
	7	0	113	104	97	112	102	103	108	119	111	109	97	94	87	93	103	150	137	150	152	161	150	190	173	174	175	194	143	139	134	120	124	116	123	98	80	91	2
	8	0	91	98	90	66	87	99	104	103	85	85	76	70	68	70	79	109	119	114	139	119	151	152	170	132	131	141	122	127	107	103	99	102	100	85	76	82	1
	9	1	67	81	74	68	73	80	79	68	73	52	68	43	62	67	67	75	76	97	105	103	133	116	104	136	119	114	91	90	95	81	70	83	81	71	82	71	3
	10	0	55	62	73	67	54	55	56	65	55	62	42	20	55	44	71	68	73	78	91	96	90	103	112	80	112	74	72	76	75	56	72	55	64	65	56	54	1
	11	0	42	51	59	44	45	42	56	48	35	52	38	21	32	53	37	50	69	61	81	71	87	115	95	69	79	77	57	61	59	69	67	67	48	51	48	56	2
	12	0	50	39	43	49	42	45	43	37	30	34	27	25	22	31	25	38	52	61	63	78	70	73	87	68	68	58	44	64	53	53	48	41	46	53	40	40	1
	13	0	25	27	35	29	39	34	26	20	22	20	21	27	18	19	22	34	27	48	65	59	58	64	55	46	41	50	44	44	38	36	33	44	36	33	27	28	0
	14	0	34	31	28	32	40	27	30	30	25	16	22	13	20	15	17	25	40	35	45	52	56	63	57	36	32	36	46	36	30	38	35	21	30	28	21	25	5
	15	0	21	25	25	28	27	22	19	22	19	7	13	14	16	17	13	25	37	38	34	54	70	57	35	33	35	31	29	28	32	29	29	23	14	18	25	25	0
	16	0	21	15	13	26	21	26	14	13	11	13	7	13	9	16	6	16	31	26	58	55	55	49	35	24	29	31	21	27	27	26	30	23	25	29	31	12	0
	17	0	18	24	22	24	27	10	16	16	8	7	7	9	7	10	14	17	25	22	36	48	34	34	32	23	28	27	27	27	29	24	15	19	22	18	22	5	1
	18	0	13	17	14	20	19	15	9	7	10	6	7	6	6	6	8	13	21	25	33	38	29	26	36	23	17	13	15	18	24	19	17	14	12	13	14	15	1
	19	0	12	11	17	18	18	12	19	10	8	6	3	2	5	7	9	8	19	24	27	40	24	22	26	15	21	21	12	14	19	11	17	18	8	11	14	14	1
	20	0	9	12	16	21	11	6	9	12	8	5	8	1	4	5	9	11	14	17	20	24	25	22	20	20	14	12	11	18	13	11	13	11	8	6	4	7	0
	21-25	0	41	49	37	53	35	24	29	25	12	13	11	15	22	22	26	33	43	62	88	91	79	84	75	53	47	37	52	51	51	31	41	28	25	31	40	36	4
	26-30	0	14	19	22	25	21	10	10	13	10	13	7	3	6	9	8	16	19	38	38	42	49	51	34	22	16	27	25	25	23	17	15	19	16	22	20	12	3
	31-35	0	9	6	7	12	5	5	9	9	2	3	1	3	2	2	3	3	9	18	36	44	32	14	18	20	14	14	13	13	9	5	16	10	3	5	7	8	0
	36-40	0	5	5	4	3	4	4	2	6	8	1	0	0	2	1	0	2	5	9	14	22	23	14	10	4	10	8	10	5	6	7	10	5	5	4	6	3	2
	41-48	0	4	3	4	6	7	4	4	4	0	1	0	0	0	3	1	1	3	5	16	17	11	12	12	7	5	1	3	7	6	3	4	6	2	5	1	2	5
	>48	0	5	1	3	2	4	2	4	2	1	0	0	0	1	0	0	0	3	5	15	21	22	18	6	1	8	6	2	7	4	0	4	4	3	1	2	1	5
MAXIMUM PERSISTENCE		9	90	50	62	64	60	50	65	69	67	47	33	33	52	47	47	47	64	59	81	90	86	96	95	56	73	69	58	77	101	48	57	91	90	52	57	55	96

TABLE 2.3-9

A COMPARISON OF SHORT-TERM TEMPERATURE DATA AT BRAIDWOOD (1974-1976), PEORIA (1973-1975), AND DRESDEN NUCLEAR POWER STATION (1974-1976)*

	AVERAGE			MAXIMUM			MINIMUM		
NORTH	BRAIDWOOD	PEORIA	DRESDEN	BRAIDWOOD	PEORIA	DRESDEN	BRAIDWOOD	PEORIA	DRESDEN
January	24.1	26.2	23.7	59.0	59	58.9	-11.6	-18	-14.5
February	30.8	31.7	30.5	67.9	59	69.8	-9.7	-14	-7.8
March	38.5	39.9	38.5	75.8	78	75.6	6.8	6	6.0
April	49.3	49.8	49.1	82.4	82	83.2	21.2	17	16.6
May	59.9	59.9	59.5	94.5	92	93.2	32.0	34	34.4
June	69.7	69.4	69.6	93.4	94	91.0	46.4	47	49.1
July	74.1	74.3	73.6	96.4	97	95.0	43.2	48	49.8
August	71.7	72.5	71.1	92.7	94	92.8	51.6	51	51.5
September	60.9	61.7	69.9	88.9	90	88.7	33.8	31	34.8
October	52.1	54.5	51.9	86.9	85	84.8	24.4	27	23.4
November	38.6	41.7	39.7	70.3	73	72.6	1.5	15	0.9
December	27.2	28.9	27.3	65.2	66	66.9	-13.0	-12	-12.8
Year	49.7	50.7	49.6	96.4	97	95.0	-13.0	-18	-14.5

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*Values in °F

TABLE 2.3-10

A COMPARISON OF SHORT-TERM TEMPERATURE DATA AT BRAIDWOOD (1974-1976), WITH LONG-TERM TEMPERATURE DATA AT PEORIA (1966-1975) AND ARGONNE (1950-1964)*

	AVERAGE			MAXIMUM			MINIMUM		
MONTH	BRAIDWOOD	PEORIA	ARGONNE	BRAIDWOOD	PEORIA	ARGONNE	BRAIDWOOD	PEORIA	ARGONNE
January	24.1	22.6	21	59.0	66	65	-11.6	-18	-20
February	30.8	27.0	26	67.9	70	67	-9.7	-14	-16
March	38.5	38.1	33	75.8	81	79	6.8	6	-9
April	49.3	50.2	47	82.4	87	84	25.2	17	14
May	59.9	59.4	58	94.5	92	90	32.0	25	27
June	69.7	70.0	68	93.4	100	96	46.4	40	34
July	74.1	73.6	71	96.4	102	101	43.2	47	45
August	71.7	71.2	70	92.7	94	96	51.6	44	41
September	60.9	63.3	63	88.9	93	96	33.8	31	32
October	52.1	53.1	53	86.9	87	89	24.4	19	16
November	38.6	40.0	37	70.3	75	77	1.5	7	-2
December	27.2	28.9	25	65.2	71	62	-13.0	-14	-18
Year	49.7	49.8	47.7	96.4	102	101	-13.0	-18	-20

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* Values in °F.

TABLE 2.3-11

AVERAGE DAILY MAXIMUM AND MINIMUM TEMPERATURE
AT PEORIA, ILLINOIS (1966-1975)*

MONTH	AVERAGE DAILY MAXIMUM	AVERAGE DAILY MAXIMUM	RANGE
January	29.7	16.2	13.5
February	34.2	20.3	13.9
March	46.4	30.4	15.9
April	59.7	41.2	18.5
May	68.7	50.4	18.5
June	79.5	60.6	18.9
July	82.8	64.9	17.9
August	80.2	62.6	17.6
September	73.2	54.5	18.7
October	63.0	44.1	18.9
November	46.6	33.3	13.3
December	34.5	23.5	11.0
Year	58.3	41.9	16.4

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*Values in °F.

TABLE 2.3-12

RELATIVE HUMIDITY DATA FOR THE 35-FOOT LEVEL AT DRESDEN (1975-1976)*

MONTH	AVERAGE	MAXIMUM	MINIMUM
January	82.0	100.0	45.9
February	76.5	100.0	33.3
March	71.9	100.0	31.3
April	64.5	98.8	17.7
May	66.2	100.0	19.6
June	68.1	100.0	24.6
July	70.1	100.0	28.7
August	71.9	100.0	20.6
September	68.9	100.0	22.0
October	66.4	100.0	19.4
November	68.9	100.0	27.3
December	78.8	100.0	39.8
Year	71.2	100.0	17.7

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*Values in °F.

TABLE 2.3-13

RELATIVE HUMIDITY DATA FOR PEORIA (1966-1975) AND ARGONNE (1950-1964)*

			PEORIA			
	AVERAGE		AVERAGE DAILY	AVERAGE DAILY	ABSOLUTE	ABSOLUTE
MONTH	PEORIA	ARGONNE	MAXIMUM	MINIMUM	MAXIMUM	MINIMUM
January	73.9	81.8	83.8	62.7	100	26
February	71.4	79.9	82.1	58.3	100	17
March	69.2	75.6	83.9	53.2	100	14
April	64.2	69.5	80.9	48.0	100	14
May	67.0	68.7	85.0	48.9	100	18
June	67.9	71.5	84.5	50.6	100	27
July	70.1	73.8	86.0	52.5	100	29
August	73.3	76.9	88.9	55.1	100	31
September	72.5	72.7	88.5	52.8	100	21
October	69.6	71.1	85.8	50.5	100	14
November	74.7	75.3	86.2	60.4	100	20
December	79.7	81.9	88.5	68.9	100	26
Year	71.2	74.9	85.4	55.2	100	14

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*Values in %.

TABLE 2.3-14

DEW-POINT TEMPERATURES FOR THE 35-FOOT LEVEL AT DRESDEN

(1975-1976)*

MONTH	AVERAGE	MAXIMUM	MINIMUM
January	18.9	57.0	-11.6
February	22.7	46.8	-9.7
March	29.8	60.5	4.4
April	36.1	65.0	9.3
May	47.7	69.8	25.9
June	59.0	75.5	37.6
July	62.2	77.1	44.7
August	61.3	75.5	39.3
September	49.6	71.4	23.6
October	38.9	62.9	16.9
November	29.1	60.8	- 1.5
December	19.6	57.4	-13.7
Year	39.5	77.1	-13.7

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*Values in °F.

TABLE 2.3-15

DEW-POINT TEMPERATURES FOR PEORIA (1966-1975) AND ARGONNE (1950-1964)*

			PEORIA			
	AVERAGE		AVERAGE DAILY	AVERAGE DAILY		
MONTH	PEORIA	ARGONNE	MAXIMUM	MINIMUM	MAXIMUM	MINIMUM
January	15.8	16.5	22.8	9.5	59	-27
February	19.0	19.8	25.0	13.1	54	-24
March	28.4	25.3	33.8	23.2	61	-11
April	37.8	36.0	43.2	32.5	70	10
May	47.3	46.0	52.0	42.3	72	16
June	57.9	56.3	62.1	53.4	79	28
July	62.4	60.7	65.8	58.6	79	41
August	61.5	60.5	64.9	57.7	77	37
September	53.6	52.3	57.7	49.1	72	25
October	42.6	41.8	47.5	37.6	66	18
November	32.0	29.3	36.7	27.5	61	0
December	23.5	19.5	28.4	18.5	57	-24
Year	40.3	38.7	45.1	35.4	79	-27

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*Values in °F.

TABLE 2.3-16

A COMPARISON OF SHORT-TERM PRECIPITATION TOTALS (WATER EQUIVALENT) AT THE BRAIDWOOD SITE (1974-1976) AND PEORIA (1974-1976)

(Values in inches)

	1974		1975		1976	
MONTH	BRAIDWOOD	PEORIA	BRAIDWOOD	PEORIA	BRAIDWOOD	PEORIA
January	2.76	3.09	2.83*	2.59	0.44*	0.78
February	1.62	1.65	1.64	2.85	2.54	2.56
March	2.31	2.69	1.93	1.73	3.06	4.25
April	3.51	4.11	3.61*	3.92	2.50	4.86
May	6.61	6.26	3.68	5.19	4.38	5.11
June	4.83	11.69	2.09*	3.90	2.78	2.92
July	1.30	2.63	1.43	4.26	2.34*	2.98
August	0.48	0.81	2.51	5.62	0.07	2.30
September	0.15	1.45	0.02	2.74	2.29	1.78
October	1.47*	2.07	1.24	3.63	1.00	2.48
November	4.23*	4.13	1.76	2.75	0.99	0.83
December	1.14	1.93	2.29	2.04	0.53	0.38

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*Data not considered reliable due to missing hours of measurement.

TABLE 2.3-17

PRECIPITATION (WATER EQUIVALENT) AVERAGES AND EXTREMES AT PEORIA (1966-1975) AND ARGONNE (1950-1964)*

	AVERAGE		MAXIMUM		MINIMUM	
MONTH	PEORIA	ARGONNE	PEORIA	ARGONNE	PEORIA	ARGONNE
January	1.55	1.42	3.09	3.52	0.56	0.03
February	1.43	1.33	2.85	2.24	0.56	0.10
March	2.28	2.19	6.95	3.85	0.93	0.23
April	3.92	3.60	7.18	5.37	0.71	1.82
May	3.83	3.08	6.26	5.55	1.30	0.13
June	4.83	3.73	11.69	7.39	0.98	1.03
July	4.64	4.32	6.04	7.05	2.63	1.29
August	2.64	3.43	5.62	6.26	0.81	1.25
September	4.55	2.81	11.49	13.17	1.45	0.86
October	3.31	2.59	5.67	13.03	0.58	0.24
November	2.18	1.72	4.13	3.53	0.79	0.86
December	2.64	1.26	4.96	2.51	1.13	0.35
Year	37.80	31.49	50.22	43.07	26.38	19.78

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*Values in inches.

TABLE 2.3-18

JOINT FREQUENCY DISTRIBUTION OF WIND DIRECTION AND PRECIPITATION OCCURRENCE FOR PEORIA (1966-1975)*

	WIND DIRECTION																	
MONTH	CLAIM	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL
January	0.2	2.1	0.7	0.7	1.0	0.8	0.6	1.3	1.3	1.9	0.5	0.6	0.8	1.5	0.6	0.8	1.2	16.8
February	0.1	2.1	0.7	0.9	1.4	1.0	0.3	1.0	0.8	1.4	0.5	0.5	0.6	0.8	1.7	1.3	1.1	15.9
March	0.1	1.5	0.9	0.9	1.7	1.7	0.4	0.7	0.8	0.8	0.4	0.8	0.5	1.0	1.5	1.0	1.2	15.8
April	0.0	0.6	0.4	0.6	0.7	1.3	1.2	0.8	0.8	2.0	0.2	0.3	0.6	0.6	0.6	0.6	0.8	12.2
May	0.1	0.4	0.3	0.7	1.0	1.0	0.5	0.7	0.9	1.5	0.5	0.6	0.5	0.6	0.6	0.5	0.2	10.5
June	0.0	0.4	0.3	0.5	0.3	0.4	0.3	0.3	0.4	1.4	0.4	0.6	0.3	0.3	0.4	0.3	0.1	6.7
July	0.2	0.4	0.2	0.3	0.6	0.3	0.1	0.2	0.3	0.8	0.3	0.2	0.3	0.2	0.2	0.2	0.2	4.9
August	0.1	0.4	0.2	0.3	0.3	0.4	0.3	0.2	0.4	1.2	0.2	0.2	0.2	0.2	0.2	0.3	0.2	5.4
September	0.2	0.7	0.9	0.8	0.5	0.5	0.5	0.3	0.8	1.4	0.3	0.6	0.1	0.5	0.2	0.2	0.5	8.9
October	0.1	0.5	0.3	0.5	0.5	0.8	0.5	0.5	1.2	1.4	0.6	0.4	0.2	0.8	0.4	0.3	0.7	9.7
November	0.1	1.6	0.6	0.9	1.1	0.5	0.4	0.6	0.7	2.1	0.5	0.4	0.8	1.5	0.8	1.4	0.8	14.7
December	0.5	1.7	1.1	0.9	1.4	2.1	1.1	1.1	2.1	2.7	0.9	0.7	1.1	1.7	1.5	1.2	1.4	23.1
Year	0.2	1.0	0.5	0.7	0.9	0.9	0.5	0.7	0.9	1.6	0.4	0.5	0.5	0.8	0.7	0.7	0.7	12.0

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*Frequency of joint occurrence in %.
Frequencies of joint occurrences are based on observations made once every 3 hours.

TABLE 2.3-19

MAXIMUM PRECIPITATION (WATER EQUIVALENT) FOR SPECIFIED TIME INTERVALS AT ARGONNE (1950-1964) AND FOR 24 HOURS AT PEORIA (1966-1975)*

	TIME INTERVAL (hr)							
	ARGONNE							PEORIA
Month	1	2	3	6	12	36	48	24
January	0.44	0.63	0.88	1.16	2.04	2.69	2.69	1.47
February	0.32	0.58	0.76	0.95	1.00	1.07	1.07	1.74
March	0.52	0.68	0.86	1.15	1.43	2.40	2.40	1.83
April	1.18	1.34	1.70	2.50	3.00	3.35	3.35	2.78
May	1.12	1.26	1.36	1.56	2.29	3.40	3.43	2.43
June	2.20	3.28	4.00	4.22	4.23	4.23	4.25	4.44
July	1.40	2.00	2.12	2.76	2.90	3.49	3.49	3.29
August	1.92	2.32	2.34	2.40	2.78	2.79	2.79	2.17
September	1.04	1.44	1.82	2.39	2.56	4.66	4.92	3.30
October	1.40	2.44	2.79	3.63	4.98	8.10	8.62	3.70
November	0.42	0.62	0.75	0.97	1.67	1.90	1.95	1.80
December	0.36	0.48	0.56	0.65	0.90	1.29	1.33	1.75
Year	2.20	3.28	4.00	4.22	4.98	8.10	8.62	4.44

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*Tabulated values in inches.

TABLE 2.3-20

ICE PELLET AND SNOW PRECIPITATION FOR PEORIA (1966-1975)*

MONTH	AVERAGE	MONTHLY MAXIMUM	24-HOUR MAXIMUM	AVERAGE NUMBER OF HOURS
January	6.3	10.2	9.0	78
February	4.0	12.8	4.7	65
March	3.7	8.3	6.0	52
April	1.0	4.6	3.6	12
May	0	0.1	0.1	1
June	0	0	0	0
July	0	0	0	0
August	0	0	0	0
September	0	0	0	0
October	0.2	1.8	1.8	2
November	3.3	9.1	6.8	41
December	5.6	18.9	10.2	83
Year	24.1	18.9	10.2	334

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*Values in inches of ice and/or snow.

TABLE 2.3-21

FREQUENCY AND PERSISTENCE OF FOG AT PEORIA (1966-1975)[*]

PERSISTENCE (hr)	JAN	FEB	MAR	APR	MAY	JUN	JULY	AUG	SEPT	OCT	NOV	DEC	YEAR
3	28	24	32	39	35	38	54	91	65	32	35	42	510
6	16	10	19	20	24	16	23	41	28	14	20	16	246
9	10	8	11	7	16	9	12	20	20	13	18	10	153
12	11	11	9	9	7	5	2	9	7	14	6	10	100
15	8	7	10	6	2	0	5	3	4	11	11	11	77
18	3	18	2	2	5	1	3	0	2	4	6	3	33
21	1	2	5	2	2	0	0	0	4	0	8	4	28
24	3	1	2	1	1	0	0	1	0	2	3	4	15
27	1	1	2	1	2	0	0	0	1	1	1	4	13
30	0	1	2	0	1	0	0	0	1	2	0	3	11
33	1	3	0	1	0	0	0	0	2	1	3	2	12
36 - 39	0	1	0	1	0	0	0	0	1	2	2	5	12
42 - 45	3	1	1	0	0	0	1	0	0	0	2	4	13
>45	5	3	2	0	0	0	0	0	0	0	0	6	18
Annual Average Total Hours of Fog	116	97	113	70	75	37	61	88	97	95	126	189	1162

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[*] Values in number of occurrences. The numbers of occurrences are based on observations made once every 3 hours, and each observation is assumed to persist for 3 hours.

TABLE 2.3-22

FOG DISTRIBUTION BY HOUR OF THE DAY AT PEORIA (1966-1975)*

HOUR OF THE DAY	WINTER	SPRING	SUMMER	FALL	YEAR
0	12.7	12.0	10.2	10.7	11.6
3	13.1	17.4	22.0	15.2	16.0
6	14.3	23.9	44.6	24.1	23.9
9	16.6	14.2	10.9	17.2	15.3
12	11.4	9.3	3.6	10.0	9.3
15	10.5	6.4	1.9	7.3	7.4
18	10.4	7.7	2.4	7.9	7.9
21	11.0	9.2	4.4	7.7	8.6

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*Values in %.

TABLE 2.3-23

FREQUENCY OF PASQUILL STABILITY CLASSES AT BRAIDWOOD (1974-1976)*

	PASQUILL STABILITY CLASS						
MONTH	A	B	C	D	E	F	G
January	1.4	1.4	1.9	49.3	34.8	8.0	3.3
February	3.7	1.3	2.6	48.9	33.8	6.6	2.9
March	5.7	2.8	4.2	41.5	37.8	6.4	1.6
April	11.0	3.0	4.7	31.4	32.2	13.2	4.7
May	5.1	1.9	3.1	32.7	40.5	11.7	4.9
June	3.8	2.8	4.2	30.3	41.0	12.7	5.3
July	7.6	7.4	7.5	29.4	25.1	15.5	7.6
August	5.2	6.1	8.6	27.7	32.9	12.2	7.2
September	4.7	3.4	3.2	17.5	41.3	17.9	12.1
October	1.7	2.3	3.1	20.0	41.4	20.2	11.4
November	0.8	0.5	1.7	29.2	50.3	12.1	5.5
December	0.5	0.8	2.9	43.8	42.6	7.6	1.9
Annual Average	4.2	2.8	3.9	32.9	37.7	12.5	6.0

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*Frequency of occurrence in % of total monthly observations.

Data for this table were derived from the three-way joint frequency distribution of wind direction, wind speed, and Pasquill stability class for the period of record.

TABLE 2.3-24

PERSISTENCE OF PASQUILL STABILITY CLASSES AT THE BRAIDWOOD SITE (1974-1976)*

PERSISTENCE (hr)	PASQUILL STABILITY CLASS A	B	C	D	E	F	G
1-3	225	482	694	1106	1149	824	222
4-6	67	17	28	352	298	197	77
7-9	38	2	1	160	168	43	51
10-12	16	0	0	101	86	19	19
13-15	0	0	0	34	62	8	8
16-18	0	0	0	26	34	0	0
19-21	0	0	0	13	10	0	0
22-24	0	0	0	5	8	0	0
25-27	0	0	0	5	2	0	0
28-30	0	0	0	2	4	1	0
31-33	0	0	0	3	2	0	0
34-39	0	0	0	2	6	0	0
40-45	0	0	0	5	3	0	0
>45	0	0	0	3	9	0	0

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*Values in number of occurrences.

TABLE 2.3-25

THREE-WAY JOINT FREQUENCY DISTRIBUTION OF WIND DIRECTION, WIND SPEED, AND PASQUILL STABILITY CLASS FOR THE 30-FOOT LEVEL AT THE BRAIDWOOD SITE (1974-1976)*

STABILITY CLASS A

Speed (mph)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	Total
Calm**																	.00
1-3	.00	.01	.00	0.1	.00	.00	.01	.03	.03	.01	.00	.00	.00	.01	.00	.00	.14
4-7	.01	.06	.11	.09	.13	.10	.13	.02	.04	.04	.02	.01	.04	.10	.09	.05	1.06
8-12	.13	.07	.05	.05	.05	.05	.15	.10	.15	.09	.10	.11	.09	.14	.26	.23	1.81
13-18	.06	.02	.00	.00	.01	.00	.02	.07	.09	.13	.12	.07	.04	.07	.10	.17	.97
19-24	.00	.00	.00	.00	.00	.00	.00	.01	.02	.01	.02	.01	.02	.03	.02	.00	.15
> 24	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.01	.01	.00	.03	.00	.00	.07
Totals	.20	.15	.18	.15	.20	.17	.31	.24	.32	.28	.27	.21	.19	.39	.47	.46	4.21

STABILITY CLASS B

Speed (mph)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	Total
Calm**																	.00
1-3	.01	.00	.00	.01	.01	.00	.01	.00	.00	.00	.00	.01	.00	.00	.00	.00	.08
4-7	.06	.03	.07	.09	.07	.04	.07	.04	.06	.08	.03	.06	.06	.06	.08	.10	.99
8-12	.13	.08	.05	.01	.03	.01	.07	.09	.14	.12	.08	.09	.06	.08	.10	.09	1.23
13-18	.00	.01	.01	.01	.00	.00	.01	.02	.03	.11	.07	.04	.01	.05	.05	.02	.47
19-24	.00	.00	.00	.00	.00	.00	.00	.00	.01	.01	.00	.00	.00	.02	.00	.00	.05
> 24	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.01
Totals	.20	.13	.14	.11	.11	.06	.16	.16	.24	.34	.19	.21	.14	.20	.22	.21	2.83

TABLE 2.3-25 (Cont'd)

STABILITY CLASS C

Speed (mph)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	Total
Calm**																	.02
1-3	.00	.01	.02	.02	.02	.01	.01	.01	.00	.02	.01	.01	.02	.01	.01	.01	.23
4-7	.09	.06	.08	.07	.09	.07	.06	.06	.09	.06	.08	.11	.12	.09	.06	.11	1.30
8-12	.13	.10	.07	.04	.02	.01	.10	.09	.11	.16	.13	.16	.10	.08	.13	.06	1.51
13-18	.05	.02	.02	.00	.00	.00	.03	.03	.04	.12	.13	.05	.06	.09	.02	.06	.72
19-24	.00	.00	.01	.00	.00	.00	.00	.00	.00	.02	.02	.01	.00	.03	.01	.00	.13
> 24	.00	.00	.00	.00	.00	.00	.00	.00	.01	.01	.01	.00	.00	.00	.00	.00	.02
Totals	.27	.19	.20	.13	.13	.09	.21	.20	.25	.40	.37	.35	.32	.30	.24	.25	3.93

STABILITY CLASS D

Speed (mph)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	Total
Calm**																	.16
1-3	.13	.11	.08	.18	.17	.15	.15	.11	.05	.07	.11	.10	.10	.12	.10	.07	1.81
4-7	.48	.57	.72	.85	.58	.43	.70	.53	.56	.45	.51	.50	.56	.55	.52	.57	9.06
8-12	.89	.92	1.09	.44	.22	.32	.63	.65	.84	.97	.76	.81	.83	1.12	1.12	.96	12.56
13-18	.40	.46	.41	.08	.01	.08	.19	.29	.62	.82	.64	.42	.48	.84	.72	.53	6.99
19-24	.09	.00	.06	.01	.00	.00	.00	.06	.34	.52	.19	.14	.20	.19	.13	.10	2.03
> 24	.00	.00	.00	.00	.00	.00	.00	.00	.03	.08	.09	.02	.00	.00	.01	.01	.24
Totals	1.99	2.05	2.36	1.56	.98	.97	1.68	1.63	2.44	2.92	2.30	1.99	2.17	2.82	2.59	2.24	32.85

TABLE 2.3-25 (Cont'd)

STABILITY CLASS E

Speed (mph)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	Total
Calm**																	.48
1-3	.17	.20	.31	.57	.57	.36	.31	.25	.20	.16	.16	.13	.17	.21	.28	.21	4.26
4-7	.78	.55	.59	.83	.88	.65	.91	.97	.97	.97	.95	.82	.78	.83	.74	.68	12.91
8-12	.53	.47	.34	.23	.18	.31	.66	1.01	1.70	1.78	1.13	.79	.63	.63	.68	.58	11.64
13-18	.21	.27	.09	.02	.00	.04	.16	.64	1.49	1.13	.60	.26	.19	.27	.15	.19	5.72
19-24	.05	.17	.08	.00	.00	.00	.01	.21	.82	.53	.10	.09	.08	.04	.01	.06	2.26
> 24	.00	.02	.00	.00	.00	.00	.00	.02	.20	.16	.02	.01	.01	.00	.00	.01	.46
Totals	1.75	1.68	1.40	1.65	1.63	1.35	2.05	3.10	5.38	4.73	2.97	2.10	1.86	1.98	1.88	1.74	37.73

STABILITY CLASS F

Speed (mph)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	Total
Calm**																	.73
1-3	.11	.12	.16	.41	.47	.41	.24	.20	.19	.18	.15	.14	.18	.20	.26	.15	3.56
4-7	.22	.17	.06	.11	.13	.36	.49	.49	.59	.81	.61	.66	.38	.41	.30	.30	6.09
8-12	.04	.02	.02	.00	.00	.01	.11	.20	.56	.44	.19	.08	.04	.06	.03	.08	1.89
13-18	.05	.00	.00	.00	.00	.00	.00	.01	.07	.00	.00	.00	.00	.00	.01	.01	.17
19-24	.00	.02	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.02
> 24	.00	.00	.00	.00	.00	.00	.00	.00	.00	.01	.00	.00	.00	.00	.00	.00	.01
Totals	.42	.33	.23	.53	.59	.79	.84	.90	1.41	1.45	.95	.88	.60	.67	.61	.54	12.47

TABLE 2.3-25 (Cont'd)

STABILITY CLASS G

Speed (mph)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	Total
Calm**																	.70
1-3	.09	.08	.10	.12	.29	.24	.21	.12	.18	.23	.12	.18	.19	.13	.12	.08	2.46
4-7	.04	.01	.02	.02	.07	.19	.24	.23	.34	.57	.25	.28	.17	.10	.03	.04	2.60
8-12	.00	.01	.00	.00	.00	.00	.00	.02	.07	.08	.01	.01	.00	.00	.00	.01	.22
13-18	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
19-24	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
> 24	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
Totals	.13	.10	.12	.14	.37	.43	.45	.38	.58	.87	.38	.47	.36	.23	.15	.13	5.98

**The calm category represents conditions with wind speeds less than 0.8 mph, which is the threshold speed for the wind speed and wind direction sensors.

TABLE 2.3-26

THREE-WAY JOINT FREQUENCY DISTRIBUTION OF WIND DIRECTION, WIND SPEED, AND PASQUILL STABILITY CLASS FOR THE 199-FOOT LEVEL AT THE BRAIDWOOD SITE (1974-1976)*

STABILITY CLASS A

Speed (mph)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	Total
Calm**																	.00
1-3	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.04
4-7	.03	.03	.05	.03	.03	.05	.04	.03	.02	.01	.00	.01	.03	.04	.03	.02	.45
8-12	.06	.09	.08	.08	.11	.10	.12	.06	.05	.08	.08	.04	.06	.10	.15	.13	1.39
13-18	.11	.08	.05	.02	.03	.05	.15	.10	.18	.10	.12	.14	.09	.08	.23	.28	1.81
19-24	.06	.01	.00	.00	.01	.01	.03	.02	.06	.06	.06	.05	.01	.04	.08	.11	.62
> 24	.00	.00	.00	.00	.00	.02	.00	.01	.03	.04	.05	.03	.02	.05	.06	.02	.34
Totals	.28	.20	.17	.14	.18	.24	.34	.22	.34	.29	.32	.27	.21	.30	.56	.57	4.64

STABILITY CLASS B

Speed (mph)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	Total
Calm**																	.00
1-3	.00	.00	.00	.00	.00	.00	.00	.00	.00	.01	.00	.00	.00	.00	.00	.00	.03
4-7	.03	.02	.03	.04	.04	.02	.05	.03	.01	.02	.03	.04	.01	.05	.05	.06	.52
8-12	.08	.04	.07	.06	.04	.03	.06	.05	.11	.07	.08	.10	.06	.05	.08	.10	1.08
13-18	.06	.06	.06	.02	.03	.03	.05	.03	.10	.12	.08	.06	.04	.06	.09	.07	.95
19-24	.01	.00	.01	.00	.01	.01	.02	.01	.01	.06	.06	.04	.00	.02	.06	.02	.34
> 24	.00	.00	.01	.00	.00	.00	.01	.00	.02	.01	.02	.02	.01	.00	.03	.01	.14
Totals	.18	.12	.17	.12	.12	.10	.19	.11	.26	.28	.28	.26	.13	.18	.30	.26	3.07

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*Values in % of total observations.

TABLE 2.3-26 (Cont'd)

STABILITY CLASS C

Speed (mph)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	Total
Calm**																	.00
1-3	.00	.00	.02	.00	.01	.00	.01	.02	.00	.00	.00	.01	.01	.00	.00	.00	.11
4-7	.07	.04	.03	.06	.05	.02	.02	.03	.04	.03	.07	.08	.09	.07	.09	.09	.87
8-12	.09	.09	.06	.06	.06	.07	.06	.04	.09	.12	.07	.12	.11	.11	.05	.10	1.30
13-18	.06	.06	.09	.03	.01	.03	.06	.05	.06	.12	.17	.10	.11	.10	.11	.09	1.24
19-24	.02	.02	.03	.00	.02	.01	.03	.03	.07	.07	.08	.06	.04	.06	.06	.05	.65
> 24	.01	.00	.01	.00	.00	.00	.01	.00	.02	.02	.04	.04	.01	.05	.03	.00	.25
Total	.26	.22	.24	.15	.14	.14	.20	.17	.28	.37	.42	.41	.36	.40	.34	.34	4.43

STABILITY CLASS D

Speed (mph)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	Total
Calm**																	.04
1-3	.03	.04	.03	.03	.03	.01	.02	.02	.04	.05	.08	.04	.06	.05	.04	.03	.60
4-7	.23	.34	.28	.31	.29	.21	.28	.25	.24	.20	.21	.23	.28	.28	.27	.27	4.17
8-12	.57	.62	.72	.73	.52	.30	.45	.41	.47	.56	.47	.53	.67	.72	.53	.55	8.84
13-18	.55	.65	.99	.54	.41	.39	.50	.47	.71	.77	.82	.62	.82	1.06	1.11	.85	11.26
19-24	.25	.28	.39	.12	.11	.15	.26	.21	.65	.67	.55	.40	.45	.70	.62	.55	6.35
> 24	.09	.03	.06	.01	.00	.06	.13	.07	.39	.53	.32	.18	.24	.27	.31	.15	2.85
Total	1.74	1.95	2.46	1.74	1.36	1.12	1.65	1.43	2.51	2.79	2.45	1.99	2.52	3.08	2.88	2.40	34.11

TABLE 2.3-26 (Cont'd)

STABILITY CLASS E

Speed (mph)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	Total
Calm**																	.06
1-3	.04	.06	.04	.03	.03	.03	.04	.03	.05	.05	.02	.01	.02	.03	.05	.04	.57
4-7	.22	.17	.14	.25	.25	.13	.18	.24	.20	.23	.17	.18	.17	.19	.28	.32	3.32
8-12	.51	.38	.41	.79	.78	.44	.46	.55	.56	.56	.50	.49	.68	.57	.63	.62	8.95
13-18	.56	.50	.39	.30	.39	.43	.67	.67	1.14	1.55	1.56	1.01	.83	.67	.84	.78	12.29
19-24	.21	.20	.08	.06	.10	.24	.35	.55	1.01	1.58	.80	.38	.30	.33	.37	.20	6.75
> 24	.04	.16	.10	.00	.00	.05	.02	.48	.91	1.11	.24	.07	.07	.11	.07	.11	3.55
Total	1.58	1.47	1.17	.144	1.56	1.32	1.72	2.52	3.87	5.08	3.30	2.14	2.07	1.19	2.24	2.07	35.48

STABILITY CLASS F

Speed (mph)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	Total
Calm**																	.06
1-3	.00	.02	.04	.01	.02	.03	.01	.01	.01	.00	.00	.01	.02	.01	.06	.02	.30
4-7	.15	.08	.08	.10	.07	.07	.05	.06	.10	.07	.06	.07	.12	.06	.14	.11	1.39
8-12	.27	.15	.14	.21	.30	.27	.20	.20	.26	.21	.17	.18	.35	.31	.31	.35	3.88
13-18	.24	.14	.07	.03	.23	.33	.36	.27	.44	.59	.65	.47	.47	.40	.34	.22	5.24
19-24	.06	.03	.01	.00	.00	.04	.05	.09	.20	.37	.22	.06	.10	.06	.04	.01	1.32
> 24	.00	.02	.00	.00	.00	.00	.00	.00	.02	.04	.01	.00	.00	.00	.00	.00	.11
Totals	.72	.44	.35	.35	.62	.73	.66	.63	1.04	1.28	1.12	.08	1.06	.84	.90	.71	12.29

TABLE 2.3-26 (Cont'd)

STABILITY CLASS G

Speed (mph)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	Total
Calm**																	.06
1-3	.02	.03	.01	.02	.04	.02	.01	.02	.01	.00	.01	.02	.03	.02	.03	.02	.32
4-7	.04	.04	.07	.04	.06	.04	.06	.04	.08	.06	.04	.06	.06	.06	.09	.04	.90
8-12	.17	.05	.06	.08	.12	.11	.12	.07	.11	.14	.17	.13	.18	.17	.15	.11	1.97
13-18	.04	.03	.01	.00	.03	.19	.21	.11	.11	.19	.36	.35	.26	.28	.11	.02	2.29
19-24	.00	.01	.01	.00	.00	.02	.07	.01	.02	.10	.11	.02	.04	.02	.00	.00	.44
> 24	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
Totals	.27	.16	.17	.14	.25	.39	.47	.26	.34	.50	.70	.59	.57	.55	.37	.20	5.98

**The calm category represents conditions with wind speeds less than 0.8 mph, which is the threshold speed for the wind speed and direction sensors.

TABLE 2.3-27

THREE-WAY JOINT FREQUENCY DISTRIBUTION
OF WIND DIRECTION, WIND SPEED, AND PASQUILL STABILITY CLASS
FOR PEORIA (1966 - 1975)

(Values in % or total observations)

WIND SPEED				WIND DIRECTION																
(METER/SEC.)			CALM	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL
	CALM		.05																	.05
	-LT-	2.0		.01	.01	.01	.00	.01	.00	.02	.00	.01	.01	.01	.01	.02	.01	.00	.00	.13
A	2.0-	6.0		.00	.01	.00	.01	.01	.01	.01	.01	.01	.00	.01	.01	.00	.01	.00	.01	.11
	-GT-	6.0		.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
	TOTAL		.05	.01	.02	.01	.01	.02	.01	.03	.01	.02	.01	.01	.02	.02	.02	.01	.01	.29
	CALM		.38																	.38
	-LT-	2.0		.08	.08	.07	.09	.12	.07	.07	.10	.16	.07	.06	.10	.11	.04	.06	.07	1.34
B	2.0-	6.0		.18	.13	.13	.13	.17	.12	.16	.15	.40	.17	.22	.17	.21	.11	.18	.15	2.79
	-GT-	6.0		.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
	TOTAL		.38	.26	.20	.20	.22	.29	.20	.23	.25	.55	.24	.28	.27	.32	.15	.24	.22	4.50
	CALM		.31																	.31
	-LT-	2.0		.11	.08	.09	.08	.08	.03	.10	.11	.21	.06	.08	.09	.09	.08	.07	.07	1.43
C	2.0-	6.0		.57	.24	.37	.34	.39	.26	.38	.55	1.66	.67	.63	.51	.53	.47	.45	.40	8.41
	-GT-	6.0		.01	.00	.01	.00	.00	.01	.02	.01	.07	.06	.07	.02	.03	.04	.03	.01	.39
	TOTAL		.31	.68	.32	.47	.41	.48	.30	.49	.67	1.94	.79	.78	.62	.64	.59	.55	.48	10.53
	CALM		.38																	.38
	-LT-	2.0		.21	.18	.13	.20	.18	.15	.19	.22	.44	.18	.19	.25	.18	.23	.14	.14	3.22
D	2.0-	6.0		2.75	1.40	1.87	2.14	2.57	1.56	2.39	2.93	6.62	1.80	1.63	1.51	2.47	2.38	2.23	1.91	38.15
	-GT-	6.0		1.07	.34	.67	.77	.66	.42	.71	1.40	3.44	.76	.78	.85	2.23	2.78	1.53	.63	19.06
	TOTAL		.38	4.03	1.92	2.67	3.11	3.42	2.14	3.28	4.54	10.51	2.74	2.60	2.61	4.87	5.39	3.91	2.68	60.81
	CALM		.00																	.00
	-LT-	2.0		.09	.05	.03	.07	.06	.07	.09	.12	.21	.08	.08	.05	.08	.05	.06	.04	1.23
E	2.0-	6.0		.55	.23	.30	.51	.58	.36	.53	.64	2.05	.46	.38	.37	.70	.57	.55	.38	9.16
	-GT-	6.0		.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
	TOTAL		.00	.64	.27	.34	.59	.64	.43	.62	.76	2.26	.53	.45	.42	.78	.62	.62	.42	10.39
	CALM		.62																	.62
	-LT-	2.0		.23	.17	.10	.16	.14	.13	.17	.19	.57	.29	.20	.25	.26	.24	.17	.15	3.42
F	2.0-	6.0		.33	.11	.17	.25	.24	.15	.25	.45	1.17	.35	.33	.30	.38	.39	.35	.23	5.44
	-GT-	6.0		.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
	TOTAL		.62	.55	.28	.27	.42	.38	.28	.41	.64	1.74	.64	.53	.56	.64	.63	.52	.38	9.49
	CALM		1.81																	1.81
	-LT-	2.0		.17	.09	.07	.11	.05	.07	.10	.10	.29	.15	.15	.20	.23	.22	.10	.09	2.18
G	2.0-	6.0		.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
	-GT-	6.0		.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
	TOTAL		1.81	.17	.09	.07	.11	.05	.07	.10	.10	.29	.15	.15	.20	.23	.22	.10	.09	3.99
	CALM		3.55																	3.55
	-LT-	2.0		.90	.64	.49	.71	.64	.54	.74	.84	1.88	.84	.76	.95	.96	.87	.62	.57	12.95
ALL	2.0-	6.0		4.38	2.11	2.84	3.38	3.96	2.46	3.71	4.72	11.91	3.45	3.20	2.88	4.28	3.93	3.77	3.08	64.06
	-GT-	6.0		1.08	.35	.68	.77	.67	.43	.73	1.41	3.51	.82	.85	.87	2.26	2.83	1.56	.63	19.44
	TOTAL		3.55	6.36	3.10	4.02	4.86	5.27	3.42	5.17	6.97	17.31	5.12	4.80	4.69	7.50	7.63	5.95	4.28	100.00

TABLE 2.3-28

PERSISTENCE AND FREQUENCY OF PASQUILL STABILITY CLASSES

AT PEORIA (1966-1975)

(NUMBER OF CONSECUTIVE OCCURRENCES OF 3-HOURLY OBSERVATIONS)*

PERSISTENCE	PASQUILL STABILITY CLASS						
(HOURS)	A	B	C	D	E	F	G
3	78	577	1480	1406	1392	1161	452
6	3	212	466	632	462	409	166
9	0	74	150	263	154	181	103
12	0	23	51	235	53	52	18
15	0	0	2	145	9	8	0
18	0	0	0	111	0	0	0
21	0	0	0	120	0	0	0
24	0	0	0	58	0	0	0
27	0	0	0	52	0	0	0
30	0	0	0	55	0	0	0
33	0	0	0	41	0	0	0
36-39	0	0	0	84	0	0	0
42-45	0	0	0	81	0	0	0
>45	0	0	0	266	0	0	0
Total Hours (in %)	0.3	4.5	10.5	60.8	10.4	9.5	4.0

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* Number of occurrences are based on observations made once every 3 hours, and each occurrence is assumed to persist for 3 hours.

TABLE 2.3-29

CUMULATIVE FREQUENCY DISTRIBUTION OF X/Q^a FOR A 1-HOUR TIME PERIOD AT THE MINIMUM EXCLUSION AREA BOUNDARY DISTANCE (485 M), BRAIDWOOD SITE

CHI/Q RANGE			DOWNWIND SECTOR																
GT		LE	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	ALL
6.69-04	to		142	198	105	121	152	74	96	62	54	59	67	138	179	144	160	100	1851
			6.3	8.4	6.6	9.1	12.3	5.3	7.2	5.2	5.1	5.9	6.7	14.4	20.0	17.3	12.9	7.1	8.8
6.02-04	to	6.69-04	0	1	1	0	2	3	1	1	2	1	3	1	2	1	2	0	21
			6.3	8.5	6.7	9.1	12.4	5.5	7.3	5.3	5.3	6.0	7.0	14.5	20.2	17.4	13.1	7.1	8.9
5.42-04	to	6.02-04	8	21	5	6	0	1	0	0	1	0	0	0	0	0	3	9	54
			6.6	9.4	7.0	9.5	12.4	5.6	7.3	5.3	5.4	6.0	7.0	14.5	20.2	17.4	13.3	7.8	9.1
4.88-04	to	5.42-04	24	30	26	24	18	27	32	18	10	11	16	37	49	55	23	30	430
			7.7	10.7	8.6	11.4	13.9	7.5	9.7	6.8	6.3	7.1	8.6	18.4	25.7	24.0	15.2	9.9	11.2
4.39-04	to	4.88-04	18	16	9	9	9	17	17	8	15	13	22	34	37	27	18	15	284
			8.5	11.3	9.2	12.0	14.6	8.7	10.9	7.5	7.8	8.4	10.9	21.9	29.8	27.3	16.6	11.0	12.5
3.95-04	to	4.39-04	44	32	29	39	34	35	32	20	16	12	3	17	21	38	47	31	450
			10.5	12.7	11.0	15.0	17.4	11.2	13.3	9.2	9.3	9.6	11.2	23.7	32.2	31.9	20.4	13.2	14.7
3.55-04	to	3.95-04	0	0	0	0	0	0	0	0	0	1	0	0	0	0	0	0	1
			10.5	12.7	11.0	15.0	17.4	11.2	13.3	9.2	9.3	9.7	11.2	23.7	32.2	31.9	20.4	13.2	14.7
3.20-04	to	3.55-04	28	42	31	33	23	24	14	13	16	12	5	4	4	26	27	30	332
			11.7	14.5	13.0	17.4	19.2	12.9	14.4	10.3	10.8	10.9	11.7	24.1	32.6	35.0	22.6	15.3	16.3
2.88-04	to	3.20-04	30	21	34	21	29	29	48	41	26	33	40	92	90	53	54	38	679
			13.0	15.4	15.1	19.0	21.6	15.0	18.0	13.7	13.2	14.2	15.7	33.8	42.7	41.3	27.0	18.0	19.5
2.59-04	to	2.88-04	38	52	34	43	12	19	10	17	10	10	3	4	2	10	19	26	309
			14.7	17.6	17.3	22.3	22.5	16.4	18.7	15.1	14.2	15.2	16.0	34.2	42.9	42.5	28.5	19.9	20.9
2.33-04	to	2.59-04	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
			14.7	17.6	17.3	22.3	22.5	16.4	18.7	15.1	14.2	15.2	16.0	34.2	42.9	42.5	28.5	19.9	20.9
2.10-04	to	2.33-04	58	101	84	82	52	72	58	58	59	43	54	92	94	68	84	93	1152
			17.3	21.9	22.5	28.4	26.7	21.5	23.1	20.0	19.8	19.5	21.4	43.8	53.4	50.7	35.3	26.5	26.4
1.89-04	to	2.10-04	37	41	26	9	4	2	5	4	2	1	1	1	0	2	13	17	165
			18.9	23.7	24.2	29.1	27.1	21.7	23.4	20.4	20.0	19.6	21.5	43.9	53.4	51.0	36.3	27.7	27.2
1.70-04	to	1.89-04	71	60	56	52	54	45	45	38	53	30	37	49	60	28	50	69	797
			22.1	26.2	27.7	33.0	31.4	24.9	26.8	23.5	25.0	22.6	25.2	49.0	60.1	54.3	40.4	32.6	81.0

TABLE 2.3-29 (Cont'd)

CHI/Q RANGE			DOWNWIND SECTOR																
GT		LE	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	ALL
1.53-04	to	1.70-04	115	86	83	66	67	77	62	55	61	55	56	90	72	48	82	76	1151
			27.2	29.9	32.9	38.0	36.8	30.4	31.4	28.2	30.7	28.2	30.9	58.4	68.2	60.1	47.0	38.0	36.4
1.38-04	to	1.53-04	17	7	0	0	0	1	0	3	0	0	1	0	0	0	1	4	34
			27.9	30.2	32.9	38.0	36.8	30.5	31.4	28.4	30.7	28.2	31.0	58.4	68.2	60.1	47.1	38.3	36.6
1.24-04	to	1.38-04	101	96	85	69	78	86	64	75	65	59	51	75	62	57	81	67	1171
			32.4	34.3	38.3	43.2	43.1	36.6	36.2	34.7	36.9	34.1	36.1	66.2	75.1	66.9	53.6	43.1	42.2
1.12-04	to	1.24-04	97	88	65	63	24	38	40	38	44	21	19	15	21	23	34	53	683
			36.7	38.0	42.4	47.9	45.1	39.3	39.2	37.9	41.1	36.2	38.0	67.8	77.4	69.7	56.4	46.8	45.4
1.00-04	to	1.12-04	107	96	86	58	66	53	64	62	49	60	67	50	31	47	86	77	1059
			41.4	42.1	47.8	52.3	50.4	43.1	44.0	43.1	45.7	42.2	44.7	73.0	80.9	75.4	63.3	52.3	50.4
9.03-05	to	1.00-04	106	119	69	66	65	55	61	61	55	54	63	51	31	33	70	83	1042
			46.1	47.2	52.1	57.2	55.7	47.0	48.6	48.3	50.9	47.6	51.1	78.4	84.4	79.3	69.0	58.2	55.4
8.13-05	to	9.03-05	67	73	52	21	22	20	21	18	16	18	10	8	2	10	20	36	414
			49.1	50.3	55.4	58.8	57.4	48.5	50.1	49.8	52.4	49.4	52.1	79.2	84.6	80.5	70.6	60.8	57.4
7.32-05	to	8.13-05	88	98	59	63	68	59	56	59	69	52	73	39	22	27	50	75	957
			53.0	54.5	59.1	63.5	62.9	52.7	54.3	54.8	58.9	54.6	59.4	83.3	87.0	83.8	74.6	66.1	61.9
6.59-05	to	7.32-05	169	146	97	67	66	75	63	56	64	57	60	30	12	23	50	89	1124
			60.5	60.7	65.2	68.6	68.3	58.0	59.1	59.5	65.0	60.3	65.4	86.4	88.4	86.5	78.7	72.5	67.2
5.93-05	to	6.59-05	79	83	62	44	54	66	63	52	55	53	52	15	12	22	33	47	792
			64.0	64.2	69.1	71.9	72.6	62.8	63.8	63.8	70.2	65.6	70.7	88.0	89.7	89.2	81.3	75.8	71.0
5.33-05	to	5.93-05	138	110	65	44	34	77	54	55	45	68	54	22	8	8	34	54	870
			70.1	68.9	73.2	75.2	75.4	68.3	67.8	68.5	74.5	72.4	76.1	90.3	90.6	90.1	84.1	79.6	75.1
4.80-05	to	5.33-05	152	149	85	67	59	99	110	83	58	94	76	13	11	17	41	83	1197
			76.9	75.3	78.6	80.2	80.1	75.3	76.0	75.4	79.9	81.9	83.7	91.6	91.8	92.2	87.4	85.6	80.8
4.32-05	to	4.80-05	104	86	54	29	31	36	35	40	34	43	32	11	3	4	12	35	589
			81.5	78.9	82.0	82.4	82.6	77.9	78.7	78.8	83.2	86.2	86.9	92.8	92.2	92.7	88.4	88.0	83.6
3.89-05	to	4.32-05	82	73	45	43	49	63	55	48	27	38	27	6	5	6	15	28	610
			85.1	82.1	84.8	85.6	86.6	82.4	82.8	82.8	85.7	90.0	89.6	93.4	92.7	93.4	89.6	90.0	86.5
0.00	to	3.89-05	336	421	241	191	166	246	230	204	151	100	103	63	65	55	129	140	2841
			100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0

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[a] X/Q values, expressed in (sec/m^3), are based on hourly onsite meteorological data for the period of record January 1974 - December 1976.
Key: 1.53-04 = 1.53 x 10^{-4}.

TABLE 2.3-30

CUMULATIVE FREQUENCY DISTRIBUTION OF X/Q^a FOR A 2-HOUR TIME PERIOD AT THE MINIMUM EXCLUSION AREA BOUNDARY DISTANCE (485 M), BRAIDWOOD SITE

CHI/Q RANGE			DOWNWIND SECTOR																
GT		LE	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	ALL
6.69-04	to		130	181	100	120	153	71	120	74	68	70	80	141	192	145	150	94	1889
			4.2	5.6	4.2	6.0	8.0	3.4	6.1	4.3	4.4	4.8	5.5	10.0	14.2	11.4	8.3	4.6	6.1
6.02-04	to	6.69-04	5	14	6	9	1	1	1	1	1	0	2	5	3	2	5	4	60
			4.3	6.0	4.5	6.5	8.1	3.5	6.2	4.3	4.5	4.8	5.7	10.4	14.4	11.5	8.5	4.7	6.3
5.42-04	to	6.02-04	5	3	1	4	1	1	2	2	0	0	0	6	4	3	5	7	44
			4.5	6.1	4.5	6.7	8.1	3.5	6.3	4.4	4.5	4.8	5.7	10.8	14.7	11.8	8.8	5.1	6.5
4.88-04	to	5.42-04	33	42	29	18	29	18	14	6	5	3	6	12	21	36	27	31	330
			5.5	7.4	5.8	7.6	9.7	4.4	7.0	4.8	4.8	5.0	6.1	11.7	16.3	14.6	10.3	6.6	7.6
4.39-04	to	4.88-04	12	9	9	10	7	5	10	3	1	6	10	7	6	15	17	12	139
			5.9	7.7	6.2	8.1	10.0	4.7	7.5	5.0	4.9	5.4	6.8	12.2	16.7	15.8	11.2	7.2	8.0
3.95-04	to	4.39-04	57	52	26	45	44	44	26	23	20	18	19	65	68	63	65	44	679
			7.8	9.3	7.3	10.3	12.3	6.8	8.8	6.3	6.1	6.7	8.1	16.8	21.7	20.7	14.8	9.3	10.2
3.55-04	to	3.95-04	12	12	9	12	3	10	4	3	3	5	6	10	16	15	10	6	136
			8.1	9.7	7.6	10.9	12.5	7.3	9.0	6.5	6.3	7.0	8.5	17.5	22.9	21.9	15.4	9.6	10.7
3.20-04	to	3.55-04	21	35	20	22	18	9	7	3	6	5	5	4	3	15	13	7	193
			8.8	10.7	8.5	12.1	13.4	7.7	9.4	6.6	6.7	7.4	8.9	17.8	23.1	23.1	16.1	9.9	11.3
2.88-04	to	3.20-04	12	12	18	18	8	12	8	11	9	12	10	15	21	11	16	13	206
			9.2	11.1	9.3	13.0	13.9	8.3	9.8	7.3	7.3	8.2	90.6	18.9	24.7	24.0	17.0	10.6	12.0
2.59-04	to	2.88-04	44	51	47	47	33	49	54	39	25	27	24	68	79	76	53	55	771
			10.6	12.7	11.3	15.3	15.6	10.7	12.5	9.5	8.9	10.0	11.2	23.7	30.5	29.9	19.9	13.2	14.5
2.33-04	to	2.59-04	15	34	13	18	1	13	7	16	4	2	3	9	6	2	11	16	170
			11.1	13.7	11.8	16.2	15.6	11.3	12.9	10.5	9.2	10.2	11.4	24.3	31.0	30.1	20.5	14.0	15.0
2.10-04	to	2.33-04	35	58	33	33	24	38	27	22	37	27	42	59	62	48	47	36	628
			12.2	15.5	13.2	17.9	16.9	13.2	14.3	11.7	11.6	12.0	14.3	28.5	35.6	33.9	23.1	15.7	17.1
1.89-04	to	2.10-04	66	77	54	46	55	56	49	37	34	22	11	33	44	51	75	54	764
			14.3	17.9	15.5	20.2	19.8	15.9	16.8	13.9	13.8	13.6	15.1	30.9	38.8	37.9	27.2	18.4	19.6
1.70-04	to	1.89-04	25	24	21	13	9	17	13	9	14	5	13	23	19	11	16	16	248
			15.1	18.6	16.4	20.8	20.3	16.7	17.4	14.4	14.7	13.9	16.0	32.5	40.2	38.7	28.1	19.1	20.4

TABLE 2.3-30 (Cont'd)

CHI/Q RANGE			DOWNWIND SECTOR																
GT		LE	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	ALL
1.53-04	to	1.70-04	137	108	101	79	103	90	110	89	82	68	82	135	158	110	116	127	1695
			19.5	22.0	20.7	24.8	25.7	21.1	23.0	19.5	20.0	18.6	21.7	42.1	51.9	47.4	34.5	25.3	25.9
1.38-04	to	1.53-04	40	28	19	22	13	14	14	16	17	17	10	21	12	17	26	19	305
			20.8	22.8	21.5	25.9	26.3	21.7	23.8	20.4	21.1	19.8	22.4	43.6	52.8	48.7	35.9	26.2	26.9
1.24-04	to	1.38-04	82	67	55	52	35	45	25	37	31	18	25	27	25	27	37	37	625
			23.4	24.9	23.8	28.5	28.2	23.9	25.0	22.6	23.1	21.0	24.1	45.6	54.6	50.8	37.9	28.0	28.9
1.12-04	to	1.24-04	126	137	110	110	62	97	87	86	74	56	64	87	89	72	102	111	1470
			27.5	29.1	28.5	34.1	31.4	28.6	29.5	27.5	27.9	24.8	28.5	51.7	61.2	56.5	43.6	33.4	33.7
1.00-04	to	1.12-04	98	95	78	69	44	48	47	39	54	50	38	55	49	42	59	73	938
			30.6	32.0	31.8	37.5	33.7	31.0	31.9	29.8	31.4	28.3	31.1	55.7	64.8	59.8	46.8	36.9	36.7
9.03-05	to	1.00-04	112	93	106	95	84	64	87	70	93	53	80	67	59	49	79	102	1293
			34.2	34.9	36.3	42.3	38.1	34.1	36.3	33.8	37.4	31.9	36.7	60.4	69.2	63.6	51.2	41.8	41.0
8.13-05	to	9.03-05	68	82	53	27	26	34	35	30	22	25	29	24	4	13	33	49	554
			36.4	37.4	38.6	43.6	39.5	35.7	38.1	35.6	38.8	33.7	38.7	62.1	69.5	64.7	53.0	44.2	42.8
7.32-05	to	8.13-05	186	160	144	105	119	115	104	103	102	98	109	111	84	68	123	142	1873
			42.3	42.4	44.7	48.9	45.8	41.3	43.4	41.5	45.4	40.4	46.2	70.0	75.7	70.0	59.7	51.1	48.9
6.59-05	to	7.32-05	166	136	98	78	103	85	66	67	60	62	51	24	24	45	76	74	1215
			47.7	46.6	48.8	52.8	51.2	45.4	46.7	45.4	49.3	44.7	49.8	71.7	77.5	73.5	63.9	54.7	52.8
5.93-05	to	6.59-05	106	98	77	55	72	84	73	74	63	72	65	57	47	32	64	75	1114
			51.1	49.6	52.1	55.6	54.9	49.5	50.5	49.7	53.4	49.6	54.3	75.8	80.9	76.0	67.5	58.3	56.4
5.33-05	to	5.93-05	138	160	111	91	45	81	86	77	52	56	44	29	14	24	56	84	1148
			55.5	54.5	56.8	60.2	57.3	53.4	54.8	54.1	56.8	53.5	57.3	77.9	82.0	77.9	70.5	62.4	60.2
4.80-05	to	5.33-05	170	150	121	75	99	88	96	105	75	117	96	49	38	53	102	111	1545
			60.9	59.2	62.0	63.9	62.5	57.7	59.7	60.2	61.6	61.5	63.9	81.4	84.8	82.1	76.2	67.7	65.2
4.32-05	to	4.80-05	165	158	100	92	98	89	92	77	70	94	85	43	32	36	72	118	1421
			66.2	64.0	66.2	68.6	67.6	62.0	64.4	64.6	66.1	68.0	69.8	84.4	87.1	84.9	80.1	73.5	69.8
3.89-05	to	4.32-05	156	143	89	91	112	89	67	66	69	67	74	36	21	30	49	86	1245
			71.2	68.4	70.0	73.1	73.5	66.4	67.8	68.4	70.6	72.6	74.9	87.0	88.7	87.3	82.8	77.6	73.9
0.00	to	3.89-05	898	1023	707	535	505	693	631	547	454	398	362	183	153	162	312	462	8025
			100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0

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[a] X/Q values, expressed in (sec/m^3), are based on hourly onsite meteorological data for the period of record January 1974 - December 1976.
Key: 1.53-04 = 1.53 x 10^{-4}.

TABLE 2.3-31

5% AND 50% PROBABILITY LEVEL χ/Q AT THE MINIMUM EXCLUSION AREA BOUNDARY DISTANCE (485 M)

BRAIDWOOD SITE

DOWNWIND	χ/Q[*]			
	5%		50%	
SECTOR	1 HOUR	2 HOURS	1 HOUR	2 HOURS
N	8.3	5.1	0.79	0.61
NNE	10.0	6.7	0.82	0.59
NE	8.4	5.2	0.95	0.64
ENE	11.0	6.5	1.10	0.71
E	21.0	11.0	1.00	0.67
ESE	7.9	4.1	0.78	0.59
SE	14.0	8.7	0.82	0.60
SSE	7.9	4.1	0.81	0.59
S	7.8	4.1	0.92	0.65
SSW	8.4	5.2	0.80	0.59
SW	11.0	6.6	0.92	0.66
WSW	41.0	14.0	1.70	1.20
W	24.0	20.0	2.20	1.60
WNW	19.0	11.0	2.10	1.30
NW	16.0	11.0	1.30	0.93
NNW	8.8	5.8	1.10	0.74
All Sectors	11.0	7.7	1.10	0.71

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[*] χ/Q values, expressed in (sec/m^3) x 10^{-4}, are based on hourly onsite meteorological data for the period of record January 1974 - December 1976

TABLE 2.3-32

CUMULATIVE FREQUENCY DISTRIBUTION OF X/Q[a] FOR AN 8-HOUR TIME PERIOD AT THE OUTER BOUNDARY OF THE LOW POPULATION ZONE (1811 M), BRAIDWOOD SITE

CHI/Q RANGE		DOWNWIND SECTOR																
GT	LE	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	ALL
1.40-04		78	115	86	81	141	57	143	83	87	96	93	115	186	83	131	30	1585
		1.3	1.9	1.7	1.8	3.2	1.3	3.4	2.3	2.0	3.2	3.1	3.9	6.2	2.1	3.3	.7	2.4
1.19-04 TO	1.40-04	33	23	14	13	18	14	33	9	16	4	14	18	16	16	37	7	231
		1.8	2.2	1.9	2.1	3.8	1.8	4.2	2.4	2.5	3.3	3.5	4.5	6.7	2.7	4.3	.8	2.8
1.01-04 TO	1.19-04	15	34	10	3	7	5	12	2	7	13	3	11	29	20	19	14	205
		2.1	2.8	2.1	2.2	3.7	1.7	4.5	2.5	2.7	3.7	3.6	4.9	7.7	3.4	4.7	1.1	3.1
8.58-05 TO	1.01-04	27	49	12	40	25	28	12	11	26	6	31	26	51	58	53	68	532
		2.5	7.8	2.3	3.3	4.3	2.4	4.8	2.8	3.5	3.9	4.6	5.7	9.4	5.3	6.1	2.7	4.0
7.30-05 TO	8.58-05	50	56	22	58	18	35	29	37	23	23	2	65	31	46	41	36	590
		3.4	4.6	2.8	4.4	4.7	3.2	5.5	3.8	4.2	4.7	4.7	7.9	10.4	6.9	7.1	3.5	4.8
6.20-05 TO	7.30-05	47	75	45	41	65	30	47	13	20	4	22	45	58	36	27	45	620
		4.2	5.0	3.6	5.3	6.2	3.0	5.6	4.2	4.8	4.8	5.4	9.4	12.3	8.1	7.8	4.5	5.8
5.27-05 TO	6.20-05	77	15	41	60	32	24	51	14	12	19	11	40	32	66	46	37	600
		4.8	7.0	4.4	6.6	6.8	4.4	8.1	4.5	5.2	5.4	5.8	10.8	13.4	10.3	9.0	5.3	6.7
4.48-05 TO	5.27-05	98	97	75	78	104	88	75	72	31	80	54	122	116	126	83	62	1325
		6.0	3.3	5.9	8.4	6.5	6.0	9.0	8.5	6.1	7.4	7.6	14.9	17.2	14.5	11.2	6.7	8.8
3.31-05 TO	4.48-05	94	82	72	93	33	47	36	44	28	25	16	68	56	73	65	58	898
		7.0	10.0	7.2	10.5	10.0	7.0	10.7	7.7	7.0	8.2	8.1	17.2	19.1	17.0	12.8	8.0	10.0
3.24-05 TO	3.31-05	91	104	87	70	43	63	51	52	29	22	24	65	89	87	100	74	1057
		9.3	11.7	8.9	12.2	11.0	8.5	12.0	9.2	7.9	8.0	8.9	19.4	22.1	19.9	15.4	9.7	11.7
2.75-05 TO	3.24-05	134	102	129	130	98	77	58	72	77	35	73	77	139	113	146	98	1627
		11.9	14.4	11.4	15.3	13.2	10.3	13.4	11.1	10.2	10.1	11.3	22.0	26.7	23.7	19.1	11.9	14.2
2.34-05 TO	2.75-05	211	218	142	124	128	151	86	98	107	77	80	156	223	226	195	230	2479
		15.0	18.0	14.1	18.0	16.2	16.7	15.4	13.8	13.5	12.8	14.3	27.3	34.1	31.3	24.0	17.2	18.0
1.99-05 TO	2.34-05	151	116	34	38	80	95	74	47	77	30	61	77	80	101	98	132	1338
		17.5	19.9	16.7	18.9	18.0	15.0	17.2	15.1	15.8	13.6	16.3	29.9	38.8	34.7	26.5	20.2	20.1
1.69-05 TO	1.99-05	235	234	117	187	138	152	151	130	152	93	105	134	217	193	206	187	2697
		21.3	23.7	19.1	22.6	21.1	19.0	21.1	18.0	20.4	15.7	19.7	34.4	44.0	41.2	31.7	24.4	24.2

TABLE 2.3-32 (Cont'd)

CHI/Q RANGE		DOWNWIND SECTOR																
GT	LE	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	ALL
1.44-05 TO	1.69-05	180 24.7	198 26.9	153 22.1	105 25.0	107 23.6	80 21.7	94 23.3	95 21.3	95 23.3	64 18.8	78 22.3	112 38.2	132 48.4	72 43.0	143 35.4	149 27.7	1867 27.1
1.22-05 TO	1.44-05	332 28.7	303 31.3	218 28.0	209 23.5	227 28.7	236 27.1	222 28.6	172 25.2	154 28.0	158 24.3	183 28.3	241 46.3	178 54.3	222 51.0	271 42.2	252 33.3	3573 32.6
1.04-05 TO	1.22-05	320 35.0	714 34.0	206 30.0	155 33.0	157 32.6	156 30.7	167 22.5	142 30.1	106 31.2	93 27.4	101 31.6	119 50.3	91 57.3	110 54.7	174 46.6	254 39.0	2668 35.7
8.82-06 TO	1.04-05	300 40.0	218 40.9	220 34.0	143 36.0	126 35.1	140 34.2	114 35.4	91 32.5	131 36.2	128 31.5	107 35.2	112 54.1	67 59.5	82 57.5	134 50.0	150 42.4	2267 40.1
7.50-06 TO	8.82-06	377 45.2	761 45.5	312 40.0	280 42.0	238 40.5	252 39.8	254 41.5	238 39.2	186 40.3	207 38.4	220 42.4	177 60.1	243 67.6	215 64.7	229 55.8	260 48.2	4012 48.3
6.37-06 TO	7.50-06	359 52.1	274 30.8	287 48.1	203 46.5	250 44.8	144 43.3	185 45.9	175 44.0	130 44.5	164 44.8	161 47.7	109 63.8	85 70.4	102 68.1	198 60.8	232 53.4	2977 50.8
5.42-06 TO	6.37-06	403 53.7	336 56.2	273 50.0	271 52.0	201 48.3	244 48.9	246 51.8	208 49.7	172 49.8	203 51.5	261 56.3	191 70.2	137 75.0	118 72.1	221 66.4	249 50.0	3743 56.0
4.80-06 TO	5.42-06	359 54.4	316 51.6	258 53.3	155 57.9	243 54.0	235 54.5	195 56.0	173 54.4	242 57.1	163 56.8	152 61.3	75 72.7	70 77.3	85 74.6	145 75.1	214 67.8	3113 61.4
3.91-06 TO	4.80-06	298 56.3	345 57.0	270 50.3	253 52.7	243 50.7	276 50.7	232 62.3	182 68.4	210 63.5	217 64.0	158 65.5	165 77.3	138 81.9	156 89.2	222 75.7	275 76.6	3630 86.9
3.33-06 TO	3.91-06	366 75.2	297 71.0	259 66.9	218 67.8	246 67.5	246 63.4	169 63.3	236 66.8	165 68.6	183 70.0	201 73.1	196 83.0	122 85.9	121 84.2	156 79.7	222 75.0	3516 72.3
2.83-06 TO	3.33-06	173 78.0	207 75.2	173 69.0	176 76.5	117 70.2	123 89.9	149 70.4	119 69.1	56 70.3	72 72.4	78 75.7	18 84.5	37 87.2	35 85.4	80 81.7	125 77.8	1743 75.0
2.40-06 TO	2.83-06	247 82.1	291 79.9	301 74.9	225 76.6	256 75.0	270 75.5	215 76.5	180 74.1	154 75.2	183 78.4	119 79.6	57 86.4	74 88.6	75 87.9	140 85.3	198 82.2	2996 79.6
2.04-06 TO	2.40-06	219 85.5	239 87.9	251 79.9	145 79.8	205 80.6	176 79.5	160 78.5	139 77.9	117 78.8	124 82.5	82 82.3	70 88.8	25 90.5	61 90.6	129 88.5	145 85.5	2283 83.1
1.74-06 TO	2.04-06	102 88.7	107 85.5	165 83.0	137 83.3	143 83.9	57 80.9	109 81.2	76 80.0	101 81.9	94 85.6	79 84.9	42 90.2	24 91.3	51 92.3	64 90.1	129 83.4	1620 95.6
0.00 TO	1.74-06	714 100.0	831 100.0	837 100.0	746 100.0	708 100.0	833 100.0	782 100.0	728 100.0	597 100.0	439 100.0	459 100.0	290 100.0	263 100.0	228 100.0	3289 100.0	519 100.0	9414 100.0

[a] X/Q values, expressed in (sec/m^3), are based on hourly onsite meteorological data for the period of record January 1974 - December 1976.
Key: 1.44-05 x 1.44 x 10^{-5}

TABLE 2.3-33

CUMULATIVE FREQUENCY DISTRIBUTION OF X/Q[a] FOR A 16-HOUR TIME PERIOD

AT THE OUTER BOUNDARY OF THE LOW POPULATION ZONE (1811 M), BRAIDWOOD SITE

CHI/Q RANGE		DOWNWIND SECTOR																
GT	LE	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	ALL
4.32-05		53	85	74	32	153	14	85	48	46	78	48	150	176	78	118	13	1264
		.7	1.0	1.0	.3	2.3	.2	1.6	.9	.8	1.7	1.1	3.4	3.9	1.7	2.0	.2	1.3
3.58-05 TO	4.32-05	38	42	4	37	59	15	34	33	17	33	34	17	15	3	2	0	384
		1.2	1.5	1.0	1.8	3.2	.5	2.1	1.5	1.3	2.4	1.8	3.8	4.2	1.8	2.0	.2	1.7
3.12-05 TO	3.58-05	15	21	5	53	24	27	30	32	20	35	63	14	20	3	58	27	519
		1.3	1.7	1.1	1.8	3.6	.9	3.2	2.1	1.7	3.2	3.3	4.1	5.3	1.8	3.0	.6	2.3
2.86-05 TO	3.12-05	4	22	26	0	1	5	37	24	0	1	13	13	29	23	19	9	226
		1.4	2.0	1.4	1.3	3.6	1.0	3.8	2.5	1.7	3.2	3.6	4.4	6.0	2.3	3.3	.7	2.5
2.28-05 TO	2.86-05	35	24	47	0	33	21	16	6	30	26	18	17	63	11	51	22	420
		1.8	2.3	2.0	1.6	4.1	1.3	4.0	2.6	2.3	3.8	4.0	4.8	7.4	2.6	4.2	1.1	2.9
1.62-05 TO	2.28-05	39	70	42	79	8	41	94	18	50	25	10	40	41	39	74	23	715
		2.3	3.1	2.6	3.0	4.3	1.9	5.5	3.0	3.3	4.3	4.2	5.7	8.3	3.4	5.5	1.4	3.7
1.83-05 TO	1.82-05	44	51	4	33	6	44	9	27	15	0	23	34	42	61	28	40	471
		3.0	3.9	2.6	3.5	4.3	2.6	5.7	3.5	3.6	4.3	4.7	6.5	9.2	4.0	6.0	2.0	4.2
1.38-05 TO	1.63-05	57	87	47	50	40	33	24	42	24	23	13	69	71	40	39	40	699
		37	4.9	3.2	4.2	5.0	3.1	8.1	4.2	4.1	4.8	5.0	8.0	10.8	5.5	6.6	2.7	4.8
1.18-05 TO	1.38-05	95	121	34	86	88	73	56	60	70	37	64	45	132	118	108	132	1351
		4.8	5.3	3.9	5.6	6.3	4.3	7.0	5.4	5.6	5.7	6.4	9.1	13.7	8.2	8.5	4.7	6.3
1.00-05 TO	1.18-05	62	130	58	89	70	75	80	28	33	69	11	159	70	112	127	66	1258
		5.8	7.8	4.7	6.9	7.4	6.5	8.3	5.9	6.2	7.2	6.7	12.7	15.3	10.7	10.6	5.7	7.6
8.51-06 TO	1.00-05	109	157	80	72	64	62	86	57	54	50	27	130	72	78	91	119	1298
		7.1	0.7	6.7	8.0	6.4	6.3	8.7	8.9	7.4	8.3	7.3	15.6	16.9	12.4	12.2	7.5	0.0
7.24-06 TO	8.51-06	186	178	102	103	165	118	149	139	48	44	112	216	147	157	174	116	2196
		9.1	11.8	7.0	10.4	10.9	8.1	12.1	9.5	8.4	9.3	9.7	20.6	20.1	15.3	15.1	9.2	11.3
6.15-06 TO	7.24-06	201	223	175	180	150	118	138	142	109	69	105	154	262	273	205	154	2643
		11.4	14.5	6.3	12.8	13.2	10.0	14.4	12.1	10.6	10.8	12.1	24.1	25.9	21.8	18.6	11.6	14.0
5.23-06 TO	6.15-06	224	241	246	206	124	168	117	129	117	70	111	183	242	157	217	219	2773
		14.1	17.3	12.4	13.8	15.1	12.6	15.3	14.5	13.8	12.3	14.5	28.2	31.3	25.2	22.3	14.9	18.9

TABLE 2.3-33 (Cont'd)

CHI/Q RANGE			DOWNWIND SECTOR																
GT		LE	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	ALL
4.44-06	TO	5.23-06	377	302	165	226	154	193	138	136	214	133	145	156	259	195	246	249	3288
			18.5	20.9	14.6	19.2	17.4	15.7	18.5	17.0	17.4	15.3	17.7	31.8	37.1	29.5	26.5	13.7	20.4
3.73-06	TO	4.44-06	371	326	875	169	145	191	225	183	272	128	178	182	269	362	314	286	4007
			22.9	24.8	16.4	22.1	19.6	18.7	22.2	20.4	23.0	18.1	21.7	35.9	43.0	37.4	31.8	23.1	24.6
3.21-06	TO	3.73-06	406	405	336	230	295	354	207	206	210	209	225	298	311	321	365	385	4992
			28.6	30.3	23.6	23.6	24.1	24.9	25.5	24.2	27.3	22.7	26.5	42.7	49.9	44.4	36.5	29.9	29.8
2.73-06	TO	3.21-06	521	469	384	269	303	244	318	219	152	236	288	142	305	268	251	438	4745
			35.0	35.7	28.1	29.0	28.7	26.7	30.7	28.3	31.2	28.0	33.0	45.9	50.7	50.1	42.9	35.7	34.7
2.32-06	TO	2.73-06	536	596	463	445	388	261	351	350	249	288	298	367	255	305	439	392	5369
			41.3	42.7	34.1	35.6	34.7	32.6	36.4	34.8	36.3	33.0	39.6	54.3	62.3	56.7	50.3	41.6	41.0
1.97-06	TO	2.32-06	504	517	561	316	333	419	351	252	258	308	290	276	219	235	279	457	5392
			47.5	48.6	36.7	40.3	36.7	36.4	42.1	39.5	41.6	40.7	45.8	60.5	67.2	61.9	55.8	48.6	46.6
1.68-06	TO	1.97-06	504	444	476	352	318	411	292	243	258	259	204	129	127	223	331	324	4996
			53.3	54.1	44.6	45.6	44.6	43.0	46.8	44.0	40.9	46.4	50.5	63.5	78.8	88.7	60.6	53.5	61.7
1.42-06	TO	1.68-06	679	517	611	465	432	411	539	357	310	348	368	323	320	340	397	573	5839
			61.5	60.2	51.4	32.4	51.2	52.3	53.6	50.6	53.2	64.1	56.6	70.8	77.1	74.2	67.4	62.3	58.9
1.21-06	TO	1.42-06	401	373	606	329	245	231	211	226	214	226	134	95	73	119	173	206	3543
			66.3	57.6	56.4	57.2	56.0	56.6	63.0	54.8	57.9	59.1	61.5	72.1	78.7	76.8	70.3	65.4	62.6
1.03-06	TO	1.21-06	551	437	567	302	473	430	390	303	276	368	383	274	216	243	398	387	5928
			72.9	58.6	61.4	61.7	62.2	63.0	65.4	60.4	63.3	67.7	70.0	78.3	83.5	82.1	77.1	71.3	68.8
8.75-07	TO	1.03-06	367	353	356	316	311	316	308	310	316	140	134	171	92	142	204	245	4129
			77.1	74.0	66.1	66.3	67.0	68.8	70.3	68.2	69.7	70.6	74.1	82.2	85.6	85.2	80.6	75.0	73.1
7.44-07	TO	8.75-07	348	476	358	337	336	423	267	335	246	226	222	259	186	157	223	337	4758
			81.2	72.6	70.6	71.3	72.1	74.7	74.5	72.4	74.6	75.9	70.0	88.1	89.7	88.6	84.4	80.2	78.1
6.32-07	TO	7.44-07	326	328	341	260	362	277	254	250	136	225	142	125	92	122	119	259	3639
			85.1	83.6	76.2	75.2	77.9	76.1	78.8	77.0	77.6	80.9	82.1	90.9	91.3	91.3	86.4	84.1	81.9
5.37-07	TO	6.32-07	188	203	220	235	132	171	133	161	61	74	95	10	34	30	98	154	2000
			87.3	85.9	76.1	78.7	80.9	81.8	80.9	80.0	78.9	82.5	84.3	91.2	92.5	91.9	88.0	86.5	84.0
0.00	TO	5.37-01	1071	1105	1099	1443	1314	1150	1175	1080	1032	791	712	388	337	369	703	889	15359
			100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0

[a] X/Q values, expressed in (sec/m^3), are based on hourly onsite meteorological data for the period of record January 1974 - December 1976.
Key: 44-06 x 4.44 x 10^{-6}

TABLE 2.3-34

CUMULATIVE FREQUENCY DISTRIBUTION OF X/Q^a FOR A 72-HOUR TIME PERIOD AT THE OUTER BOUNDARY OF THE LOW POPULATION ZONE (1811 M), BRAIDWOOD SITE

CHI/Q RANGE			DOWNWIND SECTOR																
GT		LE	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	ALL
2.16-05	to		113	145	71	7	282	0	35	37	115	66	85	429	298	142	282	0	2107
			.7	.9	.5	.0	2.0	.0	.3	.3	1.0	.6	.8	4.0	2.7	1.2	2.1	.0	1.0
1.84-05	to	2.16-05	2	84	73	66	49	0	104	31	25	73	1	52	50	2	117	0	729
			.7	1.4	.9	.5	2.4	.0	1.0	.5	1.2	1.3	.8	4.5	3.2	1.3	2.9	.0	1.4
1.56-05	to	1.84-05	33	46	139	40	36	0	71	2	1	3	1	30	176	2	98	0	678
			1.0	1.7	1.8	.8	2.6	.0	1.5	.6	1.2	1.3	.8	4.8	4.8	1.3	3.6	.0	1.7
1.33-05	to	1.56-05	12	106	2	34	19	0	233	100	3	116	1	74	93	28	7	24	852
			1.0	2.4	1.8	1.0	2.8	.0	3.2	1.4	1.2	2.4	.8	5.5	5.6	1.5	3.7	.2	2.1
1.13-05	to	1.33-05	95	103	39	13	303	0	59	69	3	75	4	7	172	86	37	32	1097
			1.6	3.0	2.1	1.1	4.9	.0	3.6	1.9	1.3	3.0	.9	5.5	7.2	2.3	4.0	.4	2.6
9.59-06	to	1.13-05	82	86	33	26	111	70	169	33	74	74	178	5	109	182	50	14	1296
			2.2	3.6	2.3	1.3	5.7	.5	4.9	2.2	1.9	3.7	2.5	5.6	8.2	3.9	4.3	.5	3.2
8.16-06	to	9.59-06	174	49	30	270	91	110	116	194	98	110	200	92	90	126	7	7	1764
			3.3	3.9	2.5	3.2	6.4	1.3	5.7	3.7	2.7	4.7	4.4	6.4	9.0	5.0	4.4	.5	4.1
6.93-06	to	8.16-06	133	181	94	146	83	100	138	32	43	167	258	182	132	76	375	343	2383
			4.2	5.0	3.1	4.2	7.0	2.1	6.7	4.0	3.1	6.2	6.8	8.1	10.2	5.6	7.1	2.2	5.2
5.89-06	to	6.93-06	210	292	107	16	45	135	302	117	79	66	10	183	180	102	291	209	2344
			5.5	6.8	3.8	4.3	7.3	3.1	8.9	4.9	3.8	6.8	6.9	9.8	11.8	6.5	9.3	3.6	6.3
5.01-06	to	5.89-06	236	231	193	119	168	207	166	24	189	34	161	186	152	63	216	120	2465
			7.0	8.3	5.0	5.1	8.5	4.6	10.1	5.1	5.4	7.1	8.3	11.6	13.2	7.1	10.8	4.5	7.5
4.76-06	to	5.01-06	325	490	134	330	147	180	240	191	262	89	84	180	161	351	285	113	3562
			9.1	11.4	5.9	7.4	9.5	5.9	11.8	6.7	7.7	8.0	9.1	13.3	14.7	10.1	12.9	5.2	9.2
3.62-06	to	4.26-06	364	376	265	377	231	354	107	373	173	128	158	246	388	303	149	244	4236
			11.5	13.7	7.6	10.0	11.2	8.6	12.6	9.7	9.2	9.1	10.6	15.5	18.2	12.8	14.0	6.9	11.2
3.08-06	to	3.62-06	436	616	534	454	280	212	184	109	180	138	104	380	285	224	366	400	4902
			14.3	17.6	11.0	13.2	13.2	10.1	13.9	10.5	10.7	10.4	11.6	19.1	20.8	14.7	16.7	9.7	13.5
2.61-06	to	3.08-06	588	741	314	507	361	366	425	362	226	252	350	408	501	547	503	617	7068
			18.1	22.2	13.0	16.7	15.8	12.8	17.0	13.4	12.6	12.7	14.8	22.9	25.4	19.5	20.4	13.9	16.9

TABLE 2.3-34 (Cont'd)

CHI/Q RANGE			DOWNWIND SECTOR																
GT		LE	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	ALL
2.22-06	to	2.61-06	640	649	618	486	404	463	585	178	257	348	279	844	283	645	487	509	7675
			22.2	26.3	17.0	20.1	18.6	16.3	21.3	14.9	14.8	15.8	17.4	30.8	28.0	25.1	24.0	17.4	20.6
1.89-06	to	2.22-06	957	758	490	476	433	672	350	536	290	168	313	454	616	393	556	421	7883
			28.4	31.0	20.1	23.4	21.7	21.3	23.8	19.2	17.3	17.4	20.3	35.0	33.6	28.6	28.0	20.3	24.3
1.61-06	to	1.89-06	955	926	689	632	671	579	629	597	311	218	529	456	825	426	1014	661	10118
			34.5	36.8	24.6	27.8	26.5	25.5	28.4	24.0	20.0	19.4	25.2	39.3	41.1	32.3	35.5	24.8	29.1
1.36-06	to	1.61-06	922	779	993	641	572	764	572	538	617	586	407	725	639	1020	619	971	11365
			40.5	41.7	30.9	32.3	30.6	31.2	32.5	28.3	25.3	24.7	29.0	46.0	46.9	41.2	40.0	31.5	34.6
1.16-06	to	1.36-06	872	927	1314	817	491	529	744	515	457	472	485	422	446	355	727	832	10405
			46.1	47.5	39.4	38.0	34.1	35.1	37.9	32.4	29.2	29.0	33.5	50.0	51.0	44.3	45.3	37.2	39.4
9.86-07	to	1.16-06	1075	906	920	653	617	709	554	426	779	443	659	208	442	574	711	810	10486
			53.0	53.2	45.3	42.5	38.5	40.4	41.9	35.9	35.9	33.0	39.6	51.9	55.0	49.3	50.6	42.8	44.5
8.38-07	to	9.86-07	777	824	1240	625	930	825	823	699	846	491	611	417	747	1217	799	844	12715
			58.0	58.4	53.2	46.9	45.2	46.5	47.9	41.5	43.2	37.5	45.3	55.8	61.8	5939	56.4	48.6	50.4
7.12-07	to	8.38-07	769	937	668	744	954	800	375	716	446	604	757	642	703	587	550	1048	11300
			63.0	64.2	57.5	52.0	52.0	52.4	50.6	47.2	47.0	43.0	52.3	61.8	68.3	65.0	60.5	55.8	55.9
6.06-07	to	7.12-07	684	611	561	656	617	788	927	537	465	496	597	364	590	628	584	1136	10241
			67.4	68.1	61.1	56.6	56.4	58.2	57.3	51.5	51.0	47.5	57.9	65.2	73.6	70.5	64.7	63.6	60.8
5.15-07	to	6.06-07	580	727	783	694	789	639	836	895	593	821	492	783	334	560	1063	694	11283
			71.1	72.6	66.1	61.4	62.0	63.0	63.4	58.7	56.1	55.0	62.4	72.5	76.7	75.4	72.5	68.4	66.2
4.37-07	to	5.15-07	725	659	717	726	541	527	477	578	631	582	454	326	324	384	495	773	8919
			75.8	76.7	70.7	66.5	65.9	66.9	66.8	63.4	61.5	60.3	66.7	75.5	79.6	78.8	76.2	73.7	70.5
3.72-07	to	4.37-07	605	540	620	500	494	683	342	516	445	419	339	109	236	290	535	579	7252
			79.7	80.1	74.7	70.0	69.4	71.9	69.3	67.5	65.4	64.1	69.8	76.6	81.8	81.3	80.1	77.7	73.9
3.16-07	to	3.72-07	627	488	434	593	646	632	817	448	636	860	776	552	598	364	489	649	9609
			83.7	83.2	77.5	74.1	74.1	76.6	75.3	71.1	70.8	72.0	77.0	81.7	87.2	84.5	83.7	82.1	78.5
2.69-07	to	3.16-07	500	464	320	468	446	463	366	417	376	569	304	108	228	235	300	315	5879
			87.0	86.1	79.6	77.3	77.2	80.0	77.9	74.5	74.1	77.1	79.8	82.7	89.3	86.5	85.9	84.3	81.3
0.00	to	2.69-07	2022	2222	3184	3258	3185	2694	3048	3181	3018	2509	2171	1852	1173	1543	1929	2284	39273
			100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0

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[a] X/Q values, expressed in (sec/m^3), are based on hourly onsite meteorological data for the period of record January 1974 - December 1976.
Key: 2.22-06 = 2.22×10^{-6}.

TABLE 2.3-35

CUMULATIVE FREQUENCY DISTRIBUTION OF X/Q[a] FOR A 624-HOUR TIME PERIOD

AT THE OUTER BOUNDARY OF THE LOW POPULATION ZONE (1811 M), BRAIDWOOD SITE

CHI/Q RANGE			DOWNWIND SECTOR																
GT		LE	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	ALL
6.49-06	to		0	204	96	0	834	0	0	0	0	0	0	146	1490	381	835	0	3986
			.0	1.4	.7	.0	5.7	.0	.0	.0	.0	.0	.0	1.0	10.2	2.6	5.7	.0	1.7
5.51-06	to	6.49-06	323	453	216	0	289	0	0	49	0	39	0	759	37	111	149	0	2425
			2.2	4.5	2.1	.0	7.7	.0	.0	.3	.0	.3	.0	6.2	10.5	3.4	6.7	.0	2.7
4.69-06	to	5.51-06	298	169	170	93	130	0	190	171	0	80	36	621	187	130	305	0	2580
			4.3	5.7	3.3	.6	8.6	.0	1.3	1.5	.0	.8	.2	10.5	11.7	4.3	8.8	.0	3.9
3.98-06	to	4.69-06	74	439	224	85	24	0	428	154	157	58	36	912	367	355	485	0	3798
			4.8	8.7	4.8	1.2	8.7	.0	4.2	2.6	1.1	1.2	.5	16.7	14.3	6.7	12.2	.0	5.5
3.39-06	to	3.98-06	491	508	87	616	88	0	448	163	178	290	1	453	323	428	295	55	4424
			8.1	12.1	5.4	5.4	9.4	.0	7.3	3.7	2.3	3.2	.5	19.8	16.5	9.6	14.2	.4	7.4
2.88-06	to	3.39-06	638	558	123	671	645	0	637	74	468	200	79	420	527	283	497	371	6391
			12.5	16.0	6.3	10.0	13.8	.0	13.0	4.2	5.5	4.6	1.0	22.7	20.1	11.6	17.6	2.9	10.1
2.45-06	to	2.88-06	862	628	501	926	386	510	1119	282	146	194	134	132	528	227	730	170	7475
			18.4	20.3	9.7	16.4	16.4	3.5	20.7	6.1	6.5	5.9	2.0	23.6	23.7	13.1	22.6	4.1	13.3
2.08-06	to	2.45-06	1085	495	248	296	1311	366	1198	241	63	346	696	61	395	1083	437	260	8581
			25.8	23.7	11.4	18.4	25.4	6.0	28.9	7.8	6.9	8.3	6.7	24.0	26.4	20.5	25.6	5.9	17.0
1.77-06	to	2.08-06	1206	1192	730	640	1008	1125	1184	119	806	744	1120	694	723	879	1302	765	14237
			34.1	31.8	16.4	22.8	32.3	13.7	37.0	8.6	12.5	13.4	14.4	28.8	31.4	26.6	34.5	11.1	23.1
1.50-06	to	1.77-06	989	2230	1271	583	458	891	1044	434	772	502	998	1558	984	1004	1281	859	15858
			40.9	47.1	25.1	26.8	35.4	19.8	44.2	11.6	17.7	16.8	21.2	39.4	38.1	33.4	43.3	17.0	29.9
1.28-06	to	1.50-06	1167	2138	1650	992	1233	1064	425	1046	566	335	957	1706	1372	684	1889	2408	19632
			48.9	61.8	36.4	33.6	43.9	27.1	47.1	18.7	21.6	19.1	27.8	51.1	47.5	38.1	56.2	33.5	38.3
1.09-06	to	1.28-06	1589	1561	3186	1569	1090	1123	654	1870	332	1045	793	724	992	684	1519	1490	20221
			59.8	72.5	58.3	44.3	51.4	34.8	51.6	31.5	23.9	26.3	33.2	56.1	54.3	42.8	66.6	43.7	46.9
9.23-07	to	1.09-06	1310	1246	2043	1643	884	1301	291	746	1060	465	647	645	1017	924	1240	1565	17027
			68.7	81.0	72.3	55.6	57.4	43.7	53.6	36.6	31.2	29.4	37.7	60.5	61.3	49.1	75.1	54.4	54.2
7.84-07	to	9.23-07	1224	1127	1353	1197	1145	1708	949	990	1413	527	1292	718	927	1459	410	1462	17901
			77.1	88.7	81.5	63.8	65.3	55.4	60.1	43.4	40.8	33.1	46.5	65.4	67.6	59.1	77.9	64.4	61.9

TABLE 2.3-35 (Cont'd)

CHI/Q RANGE			DOWNWIND SECTOR																
GT		LE	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	ALL
6.67-07	to	7.84-07	1512	625	911	1555	1188	1189	1021	1489	1225	800	611	1014	874	1162	950	1869	17995
			87.5	93.0	87.8	7.5	73.4	63.6	67.1	53.6	49.2	38.5	50.7	72.4	73.6	67.1	84.4	77.2	69.6
5.67-07	to	6.67-07	493	475	434	819	1239	1157	1369	963	1126	796	530	320	564	1281	875	1272	13713
			90.9	96.3	90.7	80.1	81.9	71.5	76.4	60.2	57.0	44.0	57.3	74.6	77.5	75.9	90.4	86.0	75.5
4.82-07	to	5.67-07	521	332	244	667	1271	1211	1366	1669	1121	2246	233	312	486	1133	890	761	14463
			94.4	98.5	92.4	84.6	90.6	79.8	85.8	71.7	64.6	59.4	55.9	76.7	80.8	83.6	96.5	91.2	81.7
4.09-07	to	4.82-07	233	27	239	544	382	1130	528	456	1874	955	655	567	375	742	367	682	9764
			96.0	98.7	94.0	88.4	93.2	87.6	89.4	74.8	77.5	65.9	60.4	80.6	83.4	88.7	99.0	95.8	85.8
3.48-07	to	4.09-07	59	82	138	543	202	560	878	642	400	719	856	522	206	605	129	388	6929
			96.4	99.3	95.0	92.1	94.6	91.4	95.4	79.2	80.2	70.9	66.3	84.2	84.8	92.9	99.9	98.5	88.8
2.96-07	to	3.48-07	163	62	291	563	459	354	273	773	660	887	1004	469	1020	360	10	26	7374
			97.5	99.7	97.0	95.9	97.7	93.8	97.3	84.5	84.7	76.9	73.2	87.4	91.8	95.3	100.0	98.7	92.0
2.51-07	to	2.96-07	287	44	213	185	242	435	142	573	744	1131	815	169	213	445	0	0	5638
			99.5	100.0	98.4	97.2	99.4	96.8	98.3	88.4	89.8	84.7	78.7	88.5	93.2	98.4	100.0	98.7	94.4
2.14-07	to	2.51-07	58	0	122	118	33	32	227	536	835	409	1139	455	253	232	0	149	4598
			99.9	100.0	99.3	98.0	99.6	97.0	99.8	92.1	95.6	87.5	86.6	91.7	95.0	100.0	100.0	99.7	96.4
1.82-07	to	2.14-07	6	0	105	271	15	158	24	307	269	725	382	417	308	3	0	43	3033
			100.0	100.0	100.0	99.9	99.7	98.1	100.0	94.2	97.4	92.4	89.2	94.5	97.1	100.0	100.0	100.0	97.7
1.54-07	to	1.82-07	7	0	0	2	39	71	0	445	168	385	176	279	299	0	0	0	1871
			100.0	100.0	100.0	99.9	100.0	98.6	100.0	97.2	98.5	95.1	90.4	96.4	99.1	100.0	100.0	100.0	98.5
1.31-07	to	1.54-07	0	0	0	17	0	62	0	342	111	127	200	62	87	0	0	0	1008
			100.0	100.0	100.0	100.0	100.0	99.0	100.0	99.6	99.3	96.0	91.7	96.8	99.7	100.0	100.0	100.0	98.9
1.12-07	to	1.31-07	0	0	0	0	0	23	0	3	53	14	287	21	1	0	0	0	402
			100.0	100.0	100.0	100.0	100.0	99.2	100.0	99.6	99.7	96.1	93.7	97.0	99.7	100.0	100.0	100.0	99.1
9.48-08	to	1.12-07	0	0	0	0	0	53	0	58	23	79	121	26	40	0	0	0	400
			100.0	100.0	100.0	100.0	100.0	99.6	100.0	100.0	99.8	96.6	94.5	97.2	100.0	100.0	100.0	100.0	99.2
8.06-08	to	9.48-08	0	0	0	0	0	42	0	0	7	375	97	32	0	0	0	0	553
			100.0	100.0	100.0	100.0	100.0	99.8	100.0	100.0	99.9	99.2	95.2	97.4	100.0	100.0	100.0	100.0	99.5
0.00	to	8.06-08	0	0	0	0	0	22	0	0	18	122	700	381	0	0	0	0	1243
			100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0

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*X/Q values, expressed in (sec/m^3), are based on hourly onsite meteorological data
for the period of record January 1974 - December 1976.
Key: 6.67-07 = 6.67 x 10^{-7}.

TABLE 2.3-36

MAXIMUM χ/Q AT THE OUTER BOUNDARY OF THE LOW POPULATION ZONE (1811 METERS), BRAIDWOOD SITE

DOWNWIND	χ/Q[*]			
SECTOR	8 HOURS	16 HOURS	72 HOURS	624 HOURS
N	9.5	1.3	.32	.063
NNE	13.	1.9	.50	.079
NE	11.	1.4	.31	.086
ENE	6.3	.94	.22	.057
E	18.	3.1	.70	.14
ESE	2.8	.45	.10	.029
SE	6.7	.94	.22	.055
SSE	5.7	.85	.23	.058
S	7.6	1.0	.25	.045
SSW	6.6	.94	.24	.059
SE	20.	2.7	.61	.049
WSW	19.	3.1	.70	.16
W	18.	2.4	.60	.17
WNW	12.	1.6	.41	.072
NW	15.	2.0	.46	.088
NNW	2.7	.48	.15	.035
All sectors	20.	3.3	1.4	.69

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[*] χ/Q values, expressed in (sec/m^3) x 10^{-4}, are based on hourly onsite meteorological data for the period of record January 1974 - December 1976.

TABLE 2.3-37

FIVE % PROBABILITY LEVEL χ/Q AT THE OUTER BOUNDARY OF THE LOW POPULATION ZONE (1811 METERS), BRAIDWOOD SITE

DOWNWIND SECTOR	χ/Q^* 8 HOURS	16 HOURS	72 HOURS	624 HOURS
N	5.2	1.2	0.63	0.40
NNE	6.9	1.4	0.70	0.54
NE	5.0	0.96	0.50	0.38
ENE	6.6	1.3	0.52	0.35
E	7.2	1.4	1.1	0.73
ESE	5.0	1.1	0.48	0.23
SE	8.5	2.1	0.94	0.37
SSE	5.2	1.3	0.57	0.27
S	6.0	1.3	0.53	0.30
SSW	6.2	1.4	0.78	0.29
SW	7.2	1.4	0.77	0.22
WSW	10.	2.2	1.5	0.55
W	16.	3.3	1.5	0.94
WNW	9.0	1.5	0.80	0.41
NW	9.8	2.1	0.79	0.74
NNW	5.8	1.1	0.46	0.22
All sectors	7.1	1.4	0.71	0.41

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[*] χ/Q values, expressed in (sec/m^3) x 10^{-5}, are based on hourly onsite meteorological data for the period of record January 1974 - December 1976.

TABLE 2.3-38

FIFTY % PROBABILITY LEVEL χ/Q AT THE OUTER BOUNDARY OF LOW POPULATION ZONE (1811 METERS), BRAIDWOOD SITE

DOWNWIND	χ/Q*			
SECTOR	8 HOURS	16 HOURS	72 HOURS	624 HOURS
N	6.8	1.9	1.1	1.3
NNE	6.6	1.9	1.1	1.5
NE	5.5	1.5	0.90	1.2
ENE	5.8	1.5	0.76	1.0
E	5.2	1.5	0.75	1.1
ESE	5.2	1.5	0.76	0.85
SE	5.7	1.6	0.74	1.2
SSE	5.4	1.5	0.64	0.71
S	5.4	1.6	0.63	0.66
SSW	5.6	1.6	0.58	0.53
SW	6.1	1.7	0.75	0.69
WSW	11.	2.5	1.2	1.3
W	14.	3.2	1.2	1.2
WNW	13.	2.7	0.98	0.91
NW	8.8	2.3	1.0	1.4
NNW	7.1	1.9	0.81	0.99
All sectors	6.6	1.8	0.85	1.0

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* χ/Q values, expressed in (sec/m^3) x 10^{-6}, are based on hourly onsite meteorological data for the period of record January 1974 - December 1976.

TABLE 2.3-39

ANNUAL AVERAGE χ/Q AT THE ACTUAL BRAIDWOOD SITE BOUNDARY

DOWNWIND SECTOR	ACTUAL SITE BOUNDARY (km)	χ/Q*
N	0.61	8.5
NNE	0.91	5.0
NE	0.79	4.1
ENE	0.70	4.9
E	1.04	2.3
ESE	2.71	0.86
SE	3.41	0.65
SSE	3.44	0.57
S	4.63	0.34
SSW	0.98	1.4
SW	0.63	2.8
WSW	0.53	6.3
W	0.52	12.
WNW	0.50	11.
NW	0.50	7.8
NNW	0.51	6.7

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* χ/Q values, expressed in (sec/m^3) x 10^{-7}, are based on hourly onsite meteorological data for the period of record January 1974 - December 1976.

TABLE 2.3-40

ANNUAL AVERAGE χ/Q AT VARIOUS DISTANCES FROM THE BRAIDWOOD STATION[*]

	χ/Q									
DOWNWARD SECTOR	0.5 MILES	1.5 MILES	2.5 MILES	3.5 MILES	4.5 MILES	7.5 MILES	15 MILES	25 MILES	35 MILES	45 MILES
N	56.	14.	8.2	5.7	4.3	2.4	1.0	0.55	0.36	0.27
NNE	60.	15.	9.0	6.4	4.8	2.7	1.2	0.62	0.41	0.30
NE	40.	11.	6.7	4.8	3.6	2.0	0.89	0.48	0.32	0.23
ENE	40.	9.8	6.0	4.2	3.2	1.8	0.78	0.43	0.28	0.21
E	33.	8.9	5.6	4.0	3.1	1.8	0.79	0.43	0.29	0.22
ESE	34.	9.5	5.9	4.2	3.2	1.8	0.77	0.41	0.27	0.20
SE	31.	8.8	5.6	4.0	3.1	1.8	0.81	0.44	0.30	0.22
SSE	25.	7.6	5.0	3.6	2.8	1.6	0.72	0.39	0.26	0.19
S	19.	5.8	3.9	2.9	2.2	1.3	0.58	0.32	0.21	0.16
SSW	19.	6.0	3.9	2.9	2.2	1.2	0.55	0.29	0.19	0.14
SW	20.	6.5	4.1	2.9	2.2	1.3	0.54	0.29	0.19	0.14
WSW	33.	8.7	5.4	3.9	2.9	1.6	0.71	0.38	0.25	0.18
W	57.	13.	7.5	5.2	3.9	2.2	0.95	0.51	0.34	0.25
WNW	49.	11.	6.2	4.3	3.2	1.8	0.80	0.43	0.29	0.22
NW	36.	8.7	5.3	3.8	2.9	1.6	0.71	0.38	0.26	0.19
NNW	33.	8.6	5.3	3.7	2.8	1.6	0.70	0.38	0.25	0.18

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[*] χ/Q values, expressed in (sec/m^3) x 10^{-8}, are based on hourly onsite meteorological data for the period of record January 1974 - December 1976.

Table 2.3-41 has been deleted intentionally

TABLE 2.3-42 has been deleted intentionally.

TABLE 2.3-43

A COMPARISON OF AVERAGE MONTHLY TEMPERATURES FROM BRAIDWOOD (1974-1976) AND SPRINGFIELD (1941-1970)

	AVERAGE	
MONTH	BRAIDWOOD	SPRINGFIELD
January	24.1	26.7
February	30.8	30.4
March	38.5	39.4
April	49.3	53.1
May	59.9	63.4
June	69.7	72.9
July	74.1	76.1
August	71.7	74.4
September	60.9	67.2
October	52.1	56.6
November	38.6	41.9
December	27.2	30.5
Year	49.7	52.7

TABLE 2.3-44

1977 - TOTAL SUSPENDED PARTICULATES

(MICROGRAMS PER CUBIC METER)

		NUMBER OF SAMPLES			HIGHEST SAMPLES				ANNUAL STATISTICS	
STATION	ADDRESS	TOTAL	>150 UG M^3	>260 UG M^3	1st	2nd	3rd	4th	GEOMETRIC MEAN	STD. GEO. DEVIATION
COOK COUNTY										
Arlington Heights	33 S. Arlington Heights Rd.	115	2	0	168	152	147	146	62	1.55
Bedford Park	6535 S. Central	60	2	0	254	155	146	127	73	1.50
Bedford Park	6700 S. 78th Ave.	60	0	0	133	126	125	117	64	1.49
Blue Island	12700 Sacramento	115	11	1	282	216	205	196	78	1.55
Blue Island (RASN)	12700 Sacramento	4	0	0	117	70	57	56		
Calumet City	755 Pulaski Road	111	7	1	264	200	200	183	66	1.63
Chicago Heights	450 State Street	24	5	0	243	238	216	172		
Chicago Heights	Dixie Highway and 10th	108	3	0	191	178	175	147	56	1.65
Cicero	15th St. and 50th Avenue	101	4	0	210	172	160	155	76	1.54
Des Plaines	1755 S. Wolf Road	111	1	0	157	142	141	134	58	1.58
Evanston	1454 Elmwood	52	0	0	113	89	82	73	39	1.45
Flossmoor	999 Kedzie Avenue	107	1	0	174	144	144	138	55	1.57
Franklin Pk.	3400 N. Rose Street	113	3	0	179	160	156	138	59	1.58

TABLE 2.3-44 (Cont'd)

		NUMBER OF SAMPLES			HIGHEST SAMPLES				ANNUAL STATISTICS	
STATION	ADDRESS	TOTAL	>150 UG M^3	>260 UG M^3	1st	2nd	3rd	4th	GEOMETRIC MEAN	STD. GEO. DEVIATION
Glenview	1930 Prairie Street	30	1	0	176	125	124	120		
Harvey	157th and Lexington	106	9	0	229	221	215	207	67	1.70
Hillside	Wolf Road and Harrison	113	2	0	172	166	138	126	57	1.56
McCook	50th Street and Glencoe	56	15	0	209	187	181	179	110	1.53
McCook	Route 66 and Lawndale	50	10	0	219	217	199	187	101	1.53
Midlothian	15202 Crawford Avenue	116	3	0	158	154	152	142	49	1.58
Morton Grove	9111 Waukegan	111	2	0	169	151	145	125	58	1.62
Niles	8955 Greenwood Avenue	106	1	0	201	141	140	127	56	1.62
Oak Park	Lake and Grove St.	112	0	0	137	122	111	107	53	1.52
Orland Park	133rd and LaGrange	109	1	0	177	142	135	134	52	1.67
Palatine	1000 Quentin Road	114	3	0	162	156	152	138	49	1.64
Park Forest	100 Park Avenue	109	1	0	155	134	127	114	48	1.56
River Forest	Lathrop and Oak Avenue	115	1	0	173	134	130	128	55	1.58
Skokie	4401 Dempster	54	1	0	208	125	109	93	48	1.82
Skokie	7701 Lincoln	56	0	0	140	113	110	106	55	1.57
Summit	60th and 74th Avenue	111	11	0	196	186	172	163	78	1.63

TABLE 2.3-44 (Cont'd)

		NUMBER OF SAMPLES			HIGHEST SAMPLES				ANNUAL STATISTICS	
STATION	ADDRESS	TOTAL	>150 UG M^3	>260 UG M^3	1st	2nd	3rd	4th	GEOMETRIC MEAN	STD. GEO. DEVIATION
Wilmette	9th Street and Central Avenue	113	0	0	142	124	112	102	43	1.71
Winnetka	112 Willow	58	0	0	111	89	84	81	39	1.65
Chicago:										
Addams Elementary School	10810 S. Avenue "H"	114	33	8	385	351	342	329	118	1.69
Anthony Elementary School	9800 S. Torrence Ave.	109	17	1	313	248	232	230	88	1.65
Austin West High School	118 North Central	114	10	0	238	187	173	170	80	1.70
Calumet High Sch.	8131 South May St.	112	8	1	278	255	213	175	69	1.60
Carver High Sch.	801 East 133rd Pl.	108	13	2	292	285	235	232	83	1.62
CAMP	445 Plymouth Court	7	2	0	202	174	115	98		
Central Office Building	320 North Clark	4	0	0	101	78	74	44		
Chicago Vocational H.S.	2100 E. 87th Street	105	8	2	282	281	200	177	75	1.69

TABLE 2.3-44 (Cont'd)

STATION	ADDRESS	NUMBER OF SAMPLES TOTAL	>150 UG M^3	>260 UG M^3	HIGHEST SAMPLES 1st	2nd	3rd	4th	ANNUAL STATISTICS GEOMETRIC MEAN	STD. GEO. DEVIATION
Cooley Vocational H.S.	1225 North Sedgwick	114	19	1	373	245	235	234	95	1.57
Crib	68th St. and Lake Michigan	67	1	0	152	148	145	136	50	1.99
Edgewater	5358 North Ashland Avenue	101	4	0	218	208	167	157	65	1.62
Farr Dormitory	3300 S. Michigan Ave.	29	2	0	245	155	147	140		
Fenger Junior College	11220 South Wallace	106	12	2	291	277	245	233	72	1.66
G.S.A. Building	538 South Clark	111	6	0	225	221	183	163	78	1.59
Hale Elementary School	6140 South Melvina Ave.	111	13	3	378	287	280	250	88	1.64
Kelly High School	4136 South California	113	10	1	374	241	233	213	81	1.60
Kenwood High School	5015 Blackstone	110	9	3	309	300	261	215	66	1.76
Lakeview High School	4015 North Ashland	113	7	0	251	204	178	175	66	1.71
Lindblom High School	6130 S. Wolcott Ave.	106	6	2	288	261	230	175	77	1.58

TABLE 2.3-44 (Cont'd)

STATION	ADDRESS	NUMBER OF SAMPLES TOTAL	>150 UG M^3	>260 UG M^3	HIGHEST SAMPLES 1st	2nd	3rd	4th	ANNUAL STATISTICS GEOMETRIC MEAN	STD. GEO. DEVIATION
Logan Square	2960 W. Cortland Ave.	56	1	0	160	142	140	138	73	1.51
Medical Center	1947 W. Polk	58	5	0	177	168	164	164	79	1.53
South Water Filt. Plant	3300 E. Cheltenham	92	13	3	405	319	269	260	78	1.90
Steinmetz High School	3030 N. Mobile Avenue	111	1	0	219	139	136	133	62	1.59
Stevenson Elem. School	8010 S. Kostner Avenue	104	9	2	429	265	323	170	69	1.74
Sullivan High School	6631 N. Bosworth	106	1	0	156	146	127	121	53	1.69
Taft High School	5625 N. Natoma	100	11	1	276	220	210	207	70	1.81
Von Steuben High School	5039 N. Kimball Ave.	109	3	0	176	159	154	148	59	1.63
Washington High School	3500 E. 114th St.	113	67	19	1106	688	617	601	170	1.72
DuPAGE COUNTY										
Addison	130 W. Army Trail Rd.	56	0	0	112	108	107	100	54	1.43
Bensenville	Main and York	56	10	1	267	213	211	209	88	1.68

TABLE 2.3-44 (Cont'd)

STATION	ADDRESS	NUMBER OF SAMPLES TOTAL	>150 UG M^3	>260 UG M^3	HIGHEST SAMPLES 1st	2nd	3rd	4th	ANNUAL STATISTICS GEOMETRIC MEAN	STD. GEO. DEVIATION
Bensenville	375 Meyer	59	2	0	159	154	146	143	63	1.63
Elmhurst	118 Schiller	58	2	0	235	160	147	130	71	1.53
Naperville	175 Jackson Street	57	1	0	165	135	125	123	58	1.52
West Chicago	DuPage County Airport	55	0	0	127	119	118	109	48	1.51
West Chicago	128 W. McConnell	54	2	0	179	167	135	131	56	1.47
Wheaton	201 Reber Street	59	0	0	145	130	111	108	59	1.53
KANE COUNTY										
Elgin	1002 North Liberty	42	0	0	139	118	117	117	56	1.56
KANKAKEE COUNTY										
Bradley	610 East Liberty	51	4	1	281	206	167	166	70	1.72
KENDALL COUNTY										
Plano	Main Street	24	0	0	135	122	114	109		
LAKE COUNTY										
Island Lake	Island Lake Grade School	56	1	0	155	101	99	95	47	1.59
Lake Bluff	121 E. Sheridan	55	0	0	124	88	86	75	40	1.52

TABLE 2.3-44 (Cont'd)

STATION	ADDRESS	NUMBER OF SAMPLES TOTAL	>150 UG M^3	>260 UG M^3	HIGHEST SAMPLES 1st	2nd	3rd	4th	ANNUAL STATISTICS GEOMETRIC MEAN	STD. GEO. DEVIATION
North Chicago (RASN)	1850 Lewis Avenue	36	0	0	145	140	100	91		
Waukegan	106 Utica	60	2	0	159	151	149	148	62	1.56
Waukegan	Golf and Jackson	44	0	0	139	127	127	106	46	1.74
Waukegan	2200 Brookside	58	0	0	143	117	110	110	52	1.52
McHENRY COUNTY										
Cary	1st St. and Three Oaks Rd.	51	0	0	126	110	84	82	41	1.55
Crystal Lake	Franklin and Caroline	45	0	0	108	99	98	80	44	1.49
WILL COUNTY										
Crete	North and Elizabeth	39	0	0	108	99	98	80	--	---
Joliet	5 East Van Buren	38	1	1	368	137	128	128	--	---
Joliet	Midland and Campbell	44	0	0	146	146	139	121	--	---
Joliet	1425 North Broadway	30	2	0	163	154	139	120	--	---
Joliet	Copperfield and Briggs	29	0	0	142	81	61	59	--	---
Joliet	Joliet and Benton	56	6	0	210	200	171	163	80	1.69

TABLE 2.3-44 (Cont'd)

STATION	ADDRESS	NUMBER OF SAMPLES TOTAL	>150 UG M^3	>260 UG M^3	HIGHEST SAMPLES 1st	2nd	3rd	4th	ANNUAL STATISTICS GEOMETRIC MEAN	STD. GEO. DEVIATION
Joliet	1216 Houbolt	49	0	0	127	95	87	86	48	1.57
Joliet	501 Ella	38	3	0	165	161	158	135	80	1.51
Lockport	5th and Madison	44	2	0	164	157	150	146	63	1.64
Mokena	10940 Front Street	52	3	0	207	177	162	143	62	1.56
Monee	432 E. Main Street	40	1	0	188	140	134	109	56	1.62
Plainfield	1005 Eastern	44	3	0	165	162	156	135	59	1.62
Rockdale	Well #2 Pump Station	56	3	1	262	158	155	150	87	1.46
Romeoville	Naperville Road	44	2	0	160	155	146	135	58	1.71
Wilmington	South Joliet Street	44	0	0	123	111	110	105	54	1.51

TABLE 2.3-45

1977 - SHORT-TERM TRENDS FOR TOTAL SUSPENDED PARTICULATES

STATION	ADDRESS	ANNUAL MEAN (UG M^3)						
		1971	1972	1973	1974	1975	1976	1977
Arlington Heights	33 S. Arlington Heights Rd.	-	-	-	-	72	67	62
Bedford Park	6535 S. Central	99	105	93	104	104	78	73
Bedford Park	6700 S. 78th Ave.	83	85	88	93	79	69	64
Blue Island	12700 Sacramento	80	66	97	93	92	85	78
Calumet City	755 Pulaski Road	68	56	71	90	77	74	66
Chicago Heights	450 State Street	-	80	71	-	139	-	-
Chicago Heights	Dixie Highway and 10th	61	47	60	72	63	63	56
Cicero	15th Street and 50th Avenue	77	74	88	89	90	75	76
Des Plaines	1755 South Wolf Road	53	44	59	57	61	54	58
Evanston	1454 Elmwood	-	60	51	42	41	38	39
Flossmoor	999 Kedzie Avenue	45	50	60	63	58	66	55
Franklin Pk.	3400 North Rose Street	63	53	63	69	70	59	59
Glenview	1930 Prairie Street	65	68	61	54	-	-	-
Harvey	157th and Lexington	89	56	78	83	79	75	67
Hillside	Wolf Road and Harrison	58	59	67	74	68	68	57

TABLE 2.3-45 (Cont'd)

STATION	ADDRESS	ANNUAL MEAN (UG M^3) 1971	1972	1973	1974	1975	1976	1977
McCook	50th Street and Glencoe	114	113	125	94	104	101	110
McCook	Route 66 and Lawndale	135	120	107	96	99	91	101
Midlothian	15202 Crawford Avenue	54	42	57	68	62	59	49
Morton Grove	9111 Waukegan	70	51	61	57	-	54	58
Niles	8955 Greenwood Avenue	49	46	66	63	68	56	56
Oak Park	Lake and Grove St.	-	-	-	-	53	-	53
Orland Park	133rd and LaGrange	63	48	58	61	65	69	52
Palatine	1000 Quentin Road	47	33	43	52	51	57	49
Park Forest	100 Park Avenue	50	43	51	57	51	47	48
River Forest	Lathrop and Oak Avenue	66	54	67	68	67	59	55
Skokie	4401 Dempster	-	-	63	-	39	49	48
Skokie	7701 Lincoln	-	-	-	-	44	58	55
Summit	60th and 74th Avenue	57	62	86	87	89	88	78
Wilmette	9th Street and Central Avenue	44	38	41	54	48	42	43
Winnetka	112 Willow	53	61	36	39	38	42	39

TABLE 2.3-45 (Cont'd)

		ANNUAL MEAN (UG M^3)						
STATION	ADDRESS	1971	1972	1973	1974	1975	1976	1977
Chicago:								
Addams Elementary School	10810 South Avenue "H"	132	115	122	129	105	131	118
Anthony Elementary School	9800 South Torrence Avenue	102	97	91	95	86	90	88
Austin West High School	118 North Central	-	-	-	-	-	89	80
Calumet High School	8131 South May Street	92	80	82	80	69	73	69
Carver High School	801 East 133rd Place	108	101	92	73	73	91	83
CAMP	445 Plymouth Court	173	155	156	120	121	122	-
Central Office Building	320 Clark	115	97	82	-	-	-	
Chicago Vocational High School	2100 E. 87th Street	99	84	82	91	76	79	75
Cooley Vocational High School	1225 North Sedgwick	132	116	126	112	94	94	95
Crib	68th St. and Lake Michigan	-	-	62	52	-	-	50
Edgewater	5358 North Ashland	-	-	-	-	-	69	65
Farr Dormitory	3300 South Michigan Ave.	109	87	79	85			
Fenger Junior College	11220 South Wallace	93	79	79	85	80	80	72
G.S.A. Building	538 South Clark	116	101	108	103	95	80	78

TABLE 2.3-45 (Cont'd)

		ANNUAL MEAN (UG M^3)						
STATION	ADDRESS	1971	1972	1973	1974	1975	1976	1977
Hale Elementary School	6140 South Melvina	100	87	92	80	68	88	88
Kelly High School	4136 South California	96	96	88	90	85	83	81
Kenwood High School	5015 Blackstone	91	80	76	70	65	71	66
Lakeview High School	4015 North Ashland	93	80	83	75	70	72	66
Lindblom High School	6130 South Wolcott Ave.	83	89	80		63	75	77
Logan Square	2960 W. Cortland Ave.	104	85	78	81	72	66	73
Medical Center	1947 West Polk	122	103	127	86	90	72	79
South Water Filt. Plant	3300 E. Cheltenham	95	67	68	68	67	67	78
Steinmetz High School	3030 N. Mobile Ave.	72	67	72	65	67	64	62
Stevenson Elem. School	8010 S. Kostner Ave.	87	83	79	69	75	77	69
Sullivan High School	6631 N. Bosworth	84	71	65	63	57	58	53
Taft High School	5625 N. Natoma	76	70	76	76	60	64	70
Von Steuben High School	5039 N. Kimball Ave.	85	48	64	64	70		59
Washington High School	3500 E. 114th Street	168	134	163	153	148	175	170

TABLE 2.3-45 (Cont'd)

		ANNUAL MEAN (UG M^3)						
STATION	ADDRESS	1971	1972	1973	1974	1975	1976	1977
DuPAGE COUNTY								
Addison	130 W. Army Trail Rd.	72	91	81	66	68	53	54
Bensenville	Main and York	160	114	110	100	93	109	88
Bensenville	375 Meyer	-	-	-		55	57	63
Elmhurst	118 Schiller	94	95	69	73	69	69	71
Naperville	175 Jackson Street	-		65	69	68	58	58
West Chicago	DuPage County Airport	-		51	51	68	51	48
West Chicago	128 W. McConnell	-	-	60	58	70	59	56
Wheaton	201 Reber Street	75	76	51	40		58	59
KANE COUNTY								
Elgin	1002 North Liberty	89	92	56	57	60	59	56
KANKAKEE COUNTY								
Bradley	610 East Liberty	-	-	-	-			70
KENDALL COUNTY								
Plano	Main Street	-	-	-	-		62	

TABLE 2.3-45 (Cont'd)

		ANNUAL MEAN (UG M^3)						
STATION	ADDRESS	1971	1972	1973	1974	1975	1976	1977
LAKE COUNTY								
Island Lake	Island Lake Grade School	89	53	58	41	47	46	47
Lake Bluff	121 E. Sheridan	50	55	43	39	42	43	40
North Chicago	1850 Lewis Avenue	86	83	69	58	65	58	62
Waukegan	Golf and Jackson	-	-	-	-	-		46
Waukegan	2200 Brookside	-	-	-	-	-		52
McHENRY COUNTY								
Cary	1st St. and Three Oaks Rd.		64		60			41
Crystal Lake	Franklin and Caroline	44	53			46	49	44
WILL COUNTY								
Crete	North and Elizabeth	96	84	56	65	60	72	
Joliet	5 East Van Buren		105	81		77	91	
Joliet	Midland and Campbell	87	97	68	87	71	72	
Joliet	1425 North Broadway	98	97	78	86	78		
Joliet	Copperfield and Briggs	97	84	66	72	72	71	

TABLE 2.3-45 (Cont'd)

		ANNUAL MEAN (UG M^{3})						
STATION	ADDRESS	1971	1972	1973	1974	1975	1976	1977
Joliet	Joliet and Benton	114	95	83	78	94	83	80
Joliet	1216 Houbolt	-	-	61	59	58	75	43
Joliet	501 Ella	-	-	63	85	79	82	80
Lockport	5th and Madison	87	79	62	74	70	68	63
Mokena	10940 Front Street	87	72	61	73	61	73	62
Monee	432 E. Main Street	61	49	41	63	48	64	56
Plainfield	1005 Eastern	-	-	-			86	59
Rockdale	Well #2 Pump Station	112	104	80	112	97	104	87
Romeoville	Naperville Road	78	48	36	77	65	80	58
Wilmington	South Joliet Street	-	-	-	-	-		54

\- Site not in operation during year shown.

TABLE 2.3-46

1977 - SULFUR DIOXIDE

PARTS PER MILLION

		NUMBER OF SAMPLES				HIGHEST SAMPLES (PPM)				ANNUAL STATISTICS	
				3-HR AVG.	24-HR AVG.	3-HR AVG.		24-HR AVG.		ARITH.	STD. GEO.
STATION	ADDRESS	1 HR	24 HR	>5	>14	1ST	2ND	1ST	2ND	MEAN	DEVIATION
COOK COUNTY											
Bedford Park	6535 South Central	6647		0	0	.321	.110	.086	.046	.016	1.90
			58	NA	0	NA	NA	.017	.015	.005	2.50
Bedford Park	6700 South 78th		57	NA	0	NA	NA	.017	.016	.004	1.78
Blue Island	12700 Sacramento		116	NA	1	NA	NA	.158	.085	.007	3.65
Blue Island (RASN)	12700 Sacramento		6	NA	0	NA	NA	.049	.029		
Calumet City	755 Pulaski Road	7181		0	1	.21	.16	.15	.12	.02	2.61
			113	NA	0	NA	NA	.027	.016	.003	2.32
Chicago Heights	Dixie Highway and 10th	7504		0	0	.16	.12	.06	.06	.02	2.31
			113	NA	0	NA	NA	.030	.027	.003	2.64
Cicero	15th Street and 50th Avenue		117	NA	0	NA	NA	.063	.048	.007	3.62
Des Plaines	1755 South Wolf Road		112	NA	0	NA	NA	.028	.027	.003	2.38
Flossmoor	999 Kedzie		117	NA	0	NA	NA	.031	.025	.004	2.79
Harvey	157th and Lexington		108	NA	0	NA	NA	.038	.034	.004	2.87

TABLE 2.3-46 (Cont'd)

		NUMBER OF SAMPLES				HIGHEST SAMPLES (PPM)				ANNUAL STATISTICS	
				3-HR AVG.	24-HR AVG.	3-HR AVG.		24-HR AVG.			
STATION	ADDRESS	1 HR	24 HR	>5	>14	1ST	2ND	1ST	2ND	ARITH. MEAN	STD. GEO. DEVIATION
Hillside	Wolf Road and Harrison	7600		0	0	.17	.15	.08	.07	.02	2.31
			116	NA	0	NA	NA	.035	.033	.002	2.02
McCook	50th Street and Glencoe	7591		0	0	.138	117	.058	.058	.014	2.20
Morton Grove	9111 Waukegan		115	NA	0	NA	NA	.027	.021	.003	2.53
Oak Park	834 Lake Street		114	NA	0	NA	NA	.043	.039	.007	3.49
Park Forest	100 Park Ave.		113	NA	0	NA	NA	.042	.025	.003	2.57
Skokie	9800 Lawler	7216		0	0	.08	.08	.05	.04	.01	2.03
			114	NA	0	NA	NA	.042	.023	.003	2.26
Summit	60th and 74th Ave.		111	NA	0	NA	NA	.034	.027	.007	3.30
Wilmette	9th Street and Central Avenue		115	NA	0	NA	NA	.059	.040	.004	2.70
Chicago:											
Addams Elementary School	10810 South Ave. "H"		58	NA	0	NA	NA	.051	.044	.013	3.58

TABLE 2.3-46 (Cont'd)

		NUMBER OF SAMPLES				HIGHEST SAMPLES (PPM)				ANNUAL STATISTICS	
STATION	ADDRESS	1 HR	24 HR	3-HR AVG. >5	24-HR AVG. >14	3-HR AVG. 1ST	3-HR AVG. 2ND	24-HR AVG. 1ST	24-HR AVG. 2ND	ARITH. MEAN	STD. GEO. DEVIATION
Anthony Elementary School	9800 South Torrence		53	NA	0	NA	NA	.069	.049	.008	3.23
Austin West High School	118 North Central	1377		0	0	.06	.05	.02	.02		
			60	NA	0	NA	NA	.063	.044	.010	3.14
Calumet High School	8131 South May		61	NA	0	NA	NA	.023	.013	.004	2.64
Carver High Sch.	801 East 133rd Pl.		58	NA	0	NA	NA	.052	.039	.008	3.19
CAMP	445 Plymouth	7365		0	1	.207	.173	.142	.116	.017	-
			7	NA	0	NA	NA	.041	.030		
Cermak Pump Station	735 W. Harrison	3204		0	0	.10	.10	.08	.06		
Central Office Building	320 North Clark		0	-	-	-	-	-	-		
Chicago Vocational H.S.	2100 E. 87th St.		60	NA	0	NA	NA	.053	.051	.010	3.13
Cooley Vocational H.S.	1225 N. Sedgwick		61	NA	0	NA	NA	.083	.080	.013	4.24
Crib	68th St. and Lake Michigan		36	NA	0	NA	NA	.073	.024	.009	2.86

TABLE 2.3-46 (Cont'd)

		NUMBER OF SAMPLES				HIGHEST SAMPLES (PPM)				ANNUAL STATISTICS	
				3-HR AVG.	24-HR AVG.	3-HR AVG.		24-HR AVG.			
STATION	ADDRESS	1 HR	24 HR	>5	>14	1ST	2ND	1ST	2ND	ARITH. MEAN	STD. GEO. DEVIATION
Edgewater	5358 N. Ashland	1859		0	0	.07	.07	.04	.04		
			56	NA	0	NA	NA	.049	.024	.007	3.09
Fenger Junior College	11220 S. Wallace	2102		0	0	.07	.06	.03	.03		
			60	NA	0	NA	NA	.041	.037	.007	3.24
G.S.A. Building	538 South Clark		56	NA	0	NA	NA	.115	.092	.019	4.22
Hale Elementary School	6140 S. Melvina		61	NA	0	NA	NA	.082	.043	.013	2.96
Kelly High School	4136 S. California		60	NA	0	NA	NA	.031	.029	.006	2.91
Kenwood High Sch.	5015 Blackstone	2077		0	0	.04	.04	.03	.03		
			61	NA	0	NA	NA	.066	.064	.014	2.61
Lakeview High Sch.	4015 N. Ashland		61	NA	0	NA	NA	.078	.067	.011	3.25
Lindbloom High Sch.	6130 S. Wolcott	3777		0	0	.05	.05	.04	.04		
			60	NA	0	NA	NA	.043	.038	.007	3.26
Medical Center	1947 West Polk	7883		0	0	.214	.149	.076	.073	.012	2.13
South Water Filt. Plant	3300 E. Cheltenham		56	NA	0	NA	NA	.022	.017	.005	2.63

TABLE 2.3-46 (Cont'd)

STATION	ADDRESS	NUMBER OF SAMPLES 1 HR	24 HR	3-HR AVG. >5	24-HR AVG. >14	HIGHEST SAMPLES (PPM) 3-HR AVG. 1ST	2ND	24-HR AVG. 1ST	2ND	ANNUAL STATISTICS ARITH. MEAN	STD. GEO. DEVIATION
State Office Building	160 North LaSalle	5854		0	0	.205	.200	.122	.120		
Steinmetz High School	3030 North Mobile		59	NA	0	NA	NA	.049	.039	.010	3.14
Stevenson Elem. School	8010 S. Kostner	2015		0	0	.05	.05	.02	.02		
			57	NA	0	NA	NA	.032	.020	.006	2.94
Sullivan High School	6631 N. Bosworth		60	NA	0	NA	NA	.051	.047	.009	3.25
Taft High School	5625 N. Natoma	1813		0	0	.05	.05	.04	.04		
			56	NA	0	NA	NA	.049	.037	.007	3.27
Washington High School	3500 E. 114th St.		60	NA	0	NA	NA	.056	.037	.010	3.40
DuPAGE COUNTY											
Bensenville	375 Meyer		44	NA	0	NA	NA	.037	.024	.007	2.48
LAKE COUNTY											
Waukegan	Golf and Jackson	6641		0	0	.100	.095	.087	.081	.012	2.07
Waukegan	3010 Grand Avenue		46	NA	0	NA	NA	.042	.020	.006	2.35

TABLE 2.3-46 (Cont'd)

STATION	ADDRESS	NUMBER OF SAMPLES 1 HR	24 HR	3-HR AVG. >5	24-HR AVG. >14	HIGHEST SAMPLES (PPM) 3-HR AVG. 1ST	3-HR AVG. 2ND	24-HR AVG. 1ST	24-HR AVG. 2ND	ANNUAL STATISTICS ARITH. MEAN	STD. GEO. DEVIATION
WILL COUNTY											
Joliet	Midland and Campbell		32	NA	0	NA	NA	.023	.018		
Joliet	Joliet and Benton	8062		0	0	.172	.082	.045	.038	.008	.174
Lockport	5th and Madison		30	NA	0	NA	NA	.016	.011		
Rockdale	Well #2 Pump Station		32	NA	0	NA	NA	.025	.021		
Romeoville	Naperville Road		13	NA	0	NA	NA	.024	.013		

NA - Not Applicable.

TABLE 2.3-47

1977 - SHORT-TERM TRENDS FOR SULFUR DIOXIDE

		ANNUAL MEAN (UG M^3)						
STATION	ADDRESS	1971	1972	1973	1974	1975	1976	1977
COOK COUNTY								
Bedford Park	6535 South Central	.028	.017	.016	.021	.007	.018	.016
Bedford Park	6700 South 78th	.026	.022	.016		.006	.005	.004
Blue Island	12700 Sacramento	-	.004	.006	.016	.029	.019	.007
Calumet City	755 Pulaski	.013	.009	.009	.012	.010	.02	.02
Chicago Heights	Dixie Highway and 10th	.013	.012	.011	.012	.009	.02	.02
Cicero	15th Street and 50th Avenue	.013	.012	.012	.010	.010	.007	.007
Des Plaines	1755 South Wolf Road	-	-	.001	.002	.004	.003	.003
Flossmoor	999 Kedzie	-	-	-	-		.008	.004
Harvey	157th and Lexington	.015	.007	.008	.009	.011	.005	.004
Hillside	Wolf Road and Harrison	.009	.008	.004	.006	.006	.003	.02
McCook	50th and Glencoe	.037	.035	.035			.015	.014
Morton Grove	9111 Waukegan	.023	.006	.004	.004		.004	.003
Oak Park	834 Lake Street	-	-	-	-	-	-	.007
Park Forest	100 Park Avenue	.004	.004	.005	.004	.004	.002	.003

TABLE 2.3-47 (Cont'd)

		ANNUAL MEAN (UG M^3)						
STATION	ADDRESS	1971	1972	1973	1974	1975	1976	1977
Skokie	9800 Lawler	-	-	-	-		.02	.01
Summit	60th and 74th Avenue	.007	.007	.004	.015	.010	.007	.007
Wilmette	9th Street and Central Avenue	.020	.005	.002	.003	.005	.003	.004
Chicago:								
Addams Elementary School	10810 South Avenue "H"	-	.020	.026	.021	.016	.015	.013
Anthony Elementary School	9800 South Torrence	-	.015	.011	.003	.007	.009	.008
Austin West High School	118 North Central	-	-	-	-		.006	.010
Calumet High School	8131 South May	.015	.023	.016	.013	.009	.007	.004
Carver High School	801 East 133rd Place	.024	.022	.016	.013	.008	.010	.008
CAMP	445 Plymouth	-	-		.019	.019	.020	.017
Cermak Pump Station	735 West Harrison	-	-	-	-	-	-	
Central Office Building	320 North Clark	-	-	-	.016		.010	
Chicago Vocational High School	2100 E. 87th Street	.007	.014	.014	.912	.009	.008	.010
Cooley Vocational High School	1225 North Sedgwick	.030	.025	.022	.021	.016	.011	.013
Crib	68th and Lake Michigan	-	-	-	-	-	-	.009

TABLE 2.3-47 (Cont'd)

		ANNUAL MEAN (UG M^3)						
STATION	ADDRESS	1971	1972	1973	1974	1975	1976	1977
Edgewater	5358 North Ashland	-	-	-	-	-	-	.007
Fenger Junior College	11220 South Wallace	.017	.022	.016	.014	.011	.009	.007
G.S.A. Building	538 South Clark	.027	.036	.030	.023	.019	.013	.019
Hale Elementary School	6140 South Melvina	.017	.027	.021	.014	.011	.012	.013
Kelly High School	4136 South California	.024	.020	.014	.011	.011	.007	.006
Kenwood High School	5015 Blackstone	-	.025	.019	.014	.011	.007	.014
Lakeview High School	4015 North Ashland	.029	.027	.019	.010	.013	.009	.011
Lindbloom High School	6130 South Wolcott	.016	.018	.011	.013	.008	.007	.007
Medical Center	1947 West Polk	-	-		.031	.017	.015	.012
South Water Filt. Plant	3300 E. Cheltenham	-	.016	.015	.008	.006	.008	.005
Steinmetz High School	3030 North Mobile	.018	.014	.009	.007	.007	.006	.010
Stevenson Elem. School	8010 South Kostner	.012	.014	.019	.014	.011	.009	.006
Sullivan High School	6631 North Bosworth	.019	.022	.016	.010	.006	.007	.009
Taft High School	5625 North Natoma	.016	.017	.012	.010	.007	.006	.007
Washington High School	3500 East 114th Street	-	-	.021	.022	.013	.011	.010

\- Site not in operation during year shown.

TABLE 2.3-48

PRECIPITATION FOR AURORA AND KANKAKEE

PRECIPITATION (inches)	AURORA (1901-1962)	KANKAKEE (1917-1962)
Monthly Maximum	14.86 (October 1954)	10.69 (October 1941)
Monthly Minimum	0.08 (January 1961)	0.10 (February 1947)
24-hour Maximum	10.48 (October 1954)	8.43 (July 1957)

SNOWFALL (inches)	AURORA (1901-1962)	KANKAKEE (1917-1962)
Monthly Maximum	36.4 (December 1951)	25.7 (January 1918)

TABLE 2.3-49

MINIMUM EXCLUSION AREA BOUNDARY (MEAB) DISTANCES FOR BRAIDWOOD

SECTOR	MINIMUM EXCLUSION AREA BOUNDARY DISTANCE (feet)
N	1760
NNE	2000
NE	2165
ENE	2075
E	2075
ESE	2075
SE	2030
SSE	1875
S	1875
SSW	1875
SW	1840
WSW	1700
W	1700
WNW	1625
NW	1625
NNW	1625

BRAIDWOOD - UFSAR

TABLE 2.3-50

BRAIDWOOD STATION JOINT WIND-STABILITY CLASS FREQUENCY DISTRIBUTION

(1994-1998)

34 FT METEOROLOGICAL TOWER LEVEL

		Wind Direction Category																
Stability Class	Wind Speed Category[(1) (2)]	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	Total
1 (A)	2	0	0	2	3	0	1	3	2	0	3	0	0	1	0	0	0	15
	3	32	29	99	71	78	37	52	74	63	38	27	42	51	34	39	27	793
	4	71	77	67	12	16	20	19	35	62	108	61	71	76	82	108	106	991
	5	2	6	0	0	0	1	9	16	30	38	21	7	16	45	19	18	228
	6	0	0	0	0	0	0	0	4	9	5	1	0	0	1	0	0	20
	7	0	0	0	0	0	0	0	0	0	4	0	0	0	0	0	0	4
2 (B)	2	0	1	5	6	6	1	1	3	4	0	0	0	1	1	1	2	32
	3	31	29	51	53	53	43	48	53	30	30	30	30	62	42	42	35	662
	4	32	45	37	18	15	7	13	27	48	61	49	54	47	86	74	66	679
	5	5	4	3	0	0	0	5	14	19	26	27	12	20	30	12	15	192
	6	0	0	0	0	0	0	0	1	5	3	0	0	1	3	0	0	13
	7	0	0	0	0	0	0	0	0	1	1	0	0	0	0	0	0	2
3 (C)	2	1	3	3	12	11	6	6	2	4	2	1	3	3	1	2	0	60
	3	42	46	77	68	77	59	57	67	42	43	44	51	82	76	62	45	938
	4	41	44	47	9	16	7	23	47	54	74	81	79	78	104	106	70	880
	5	9	9	2	0	0	1	7	22	26	49	22	20	19	34	15	16	251
	6	1	0	0	0	0	0	0	2	2	11	0	1	1	5	0	0	23
	7	0	0	0	0	0	0	0	0	0	3	0	0	0	0	0	0	3
4 (D)	2	40	72	105	167	151	50	28	23	15	13	25	32	49	81	87	55	993
	3	293	348	507	601	397	294	295	326	178	164	241	364	482	464	534	436	5924
	4	259	419	452	247	98	130	236	362	330	392	490	409	546	635	437	436	5878
	5	57	129	67	5	2	20	74	176	278	361	235	142	298	309	73	148	2374
	6	9	1	0	0	0	0	0	18	67	98	40	19	46	45	2	19	364
	7	0	0	0	0	0	0	0	0	1	10	7	0	3	0	0	0	21
5 (E)	2	127	144	231	387	425	193	81	51	29	30	29	65	141	222	177	119	2451
	3	325	405	394	531	390	489	575	633	380	295	363	646	547	558	402	356	7289
	4	155	199	207	106	56	126	285	511	655	808	443	310	315	381	156	202	4915
	5	47	101	100	12	1	12	47	136	290	362	98	76	101	119	30	37	1569
	6	12	11	1	0	0	0	1	13	81	41	21	34	54	17	0	2	288
	7	0	0	0	0	0	0	0	2	5	5	8	5	7	0	0	0	32
6 (F)	2	109	75	121	189	287	207	100	59	33	44	41	85	184	281	193	120	2128
	3	66	33	14	15	34	200	151	126	73	102	96	383	303	132	54	40	1822
	4	0	0	1	0	0	0	3	18	15	114	30	16	3	1	1	1	203
	5	0	2	4	0	0	0	0	0	0	4	0	0	0	1	0	0	11
	6	0	2	5	0	0	0	0	0	0	0	0	4	0	0	0	0	11
	7	0	0	0	0	0	0	0	0	0	0	0	13	1	0	0	0	14
7 (G)	2	70	64	63	95	156	94	42	22	22	21	30	48	116	172	116	94	1225
	3	9	4	0	4	25	36	16	4	6	15	8	143	69	22	7	10	378
	4	1	0	0	0	0	0	0	1	2	6	1	1	0	0	0	0	12
	5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	6	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	7	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Notes:

1) Wind speed categories defined as follows:

Category	Wind Speed (mph)
2	≥0.8 to <3.5
3	≥3.5 to <7.5
4	≥7.5 to <12.5
5	≥12.5 to <18.5
6	≥18.5 to <24
7	≥24

2) Wind speed Category 1 is assumed for calms occurrences.
Calm occurrences by stability class: A=0, B=0, C=0, D=0, E=3, F=20, G=19

TABLE 2.3-51

BRAIDWOOD STATION JOINT WIND-STABILITY CLASS FREQUENCY DISTRIBUTION

(1994-1998)

203 FT METEOROLOGICAL TOWER LEVEL

		Wind Direction Category																
Stability Class	Wind Speed Category(1) (2)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	Total
1 (A)	2	0	0	0	1	0	0	0	0	1	0	0	0	0	0	0	0	2
	3	14	15	34	42	32	24	24	25	27	21	8	16	19	8	9	8	326
	4	54	40	71	49	50	30	30	67	56	43	45	52	77	43	61	58	826
	5	40	34	44	16	13	19	10	9	57	84	35	26	49	54	97	63	650
	6	1	2	1	2	5	4	4	6	14	38	11	5	5	34	32	9	173
	7	0	0	0	0	0	1	4	9	19	20	4	0	0	8	10	0	75
2 (B)	2	0	0	2	1	3	1	1	0	1	0	0	0	0	0	1	1	11
	3	14	14	14	37	33	29	25	21	18	16	11	20	39	18	20	14	343
	4	44	32	38	28	28	23	28	31	30	39	35	42	46	52	53	51	600
	5	17	19	29	11	18	10	8	18	36	44	39	21	25	48	60	36	439
	6	1	0	3	4	4	1	0	9	12	23	14	5	12	26	17	9	140
	7	0	0	0	0	0	0	5	5	9	9	4	0	6	5	3	0	46
3 (C)	2	0	1	1	5	6	2	3	1	1	2	0	0	2	0	1	1	26
	3	30	35	29	32	44	36	46	33	28	21	27	29	48	41	24	25	528
	4	40	35	48	40	44	32	30	37	41	43	48	67	63	78	76	58	780
	5	23	16	36	13	15	10	11	22	34	50	58	46	40	66	75	39	554
	6	3	4	5	4	1	3	8	14	25	32	12	12	9	26	18	8	184
	7	1	0	0	0	0	0	6	4	9	30	2	2	3	18	2	3	80
4 (D)	2	17	28	40	51	38	17	10	14	13	10	12	17	18	28	31	25	369
	3	168	126	188	202	225	108	135	109	97	75	119	143	170	178	251	229	2523
	4	251	249	362	407	303	175	190	256	159	136	317	322	398	431	444	333	4733
	5	168	278	419	300	178	134	150	213	270	260	360	303	410	492	400	320	4655
	6	44	86	136	62	47	68	116	127	229	285	158	77	197	282	131	100	2145
	7	10	6	10	3	4	26	51	68	144	311	74	33	105	169	38	49	1101
5 (E)	2	16	21	24	24	17	12	8	7	7	5	7	11	15	21	21	21	237
	3	140	98	156	196	154	83	99	92	72	44	107	98	88	102	135	107	1771
	4	292	253	408	534	553	254	364	362	283	225	308	411	370	421	372	380	5790
	5	147	223	268	186	186	298	370	442	581	637	481	386	407	483	308	227	5630
	6	30	61	76	31	28	73	106	187	345	520	142	67	106	161	76	36	2045
	7	20	33	55	16	1	27	45	70	191	249	58	57	109	90	29	5	1055
6 (F)	2	8	7	9	11	9	9	7	6	7	11	12	12	9	7	9	4	137
	3	57	35	39	59	44	29	49	57	48	33	55	39	32	42	50	53	721
	4	131	84	66	79	127	122	146	105	86	88	77	83	161	197	202	172	1926
	5	33	24	19	16	46	155	130	54	63	36	112	115	227	168	54	23	1275
	6	0	0	0	0	0	2	2	5	9	18	42	2	5	1	0	0	86
	7	0	1	12	0	0	0	0	0	0	6	3	9	9	1	0	0	41
7 (G)	2	9	11	12	13	7	7	11	7	10	10	15	12	9	10	9	10	162
	3	38	27	26	19	20	17	30	49	46	46	34	32	18	20	22	23	467
	4	71	34	26	20	47	49	52	34	10	12	25	26	42	69	66	69	652
	5	9	10	5	2	7	32	32	9	1	3	6	24	74	95	30	14	353
	6	0	0	0	0	0	0	2	1	0	0	4	2	7	0	0	0	16
	7	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Notes:

1) Wind speed categories defined as follows:

Category	Wind Speed (mph)
2	≥0.8 to <3.5
3	≥3.5 to <7.5
4	≥7.5 to <12.5
5	≥12.5 to <18.5
6	≥18.5 to <24
7	≥24

2) Wind speed Category 1 is assumed for calm occurrences.

Calm occurrences by stability class: A=0, B=0, C=0, D=0, E=1, F=0, G=2

TABLE 2.3-52

ARCON96 INPUT PARAMETER SUMMARY FOR BRAIDWOOD STATION

		Control Room Fresh Air Intake				**Turbine Building Emergency Air Intake**			
	ARCON96 INPUT PARAMETER	**Containment Wall**	**Plant Vent**	**PORVs/ Safety Valves**	**Main Steam Line Break**	**Containment Wall**	**Plant Vent**	**PORVs/ Safety Valves**	**Main Steam Line Break**
UNIT 1	**Release Height (m)**	**29.7**	**61**	**9.8**	**7.9**	**29.7**	**61**	**9.8**	**7.9**
	Intake Height (m)	**21.2**	**21.2**	**21.2**	**7.9**	**20.4**	**20.4**	**20.4**	**20.4**
	Horizontal Distance from Intake to Stack (m)	**7.6**	**34.1**	**22.9**	**43.3**	**30.5**	**27.4**	**35.1**	**13.4**
	Elevation Difference between Stack Grade and Intake Grade (m)	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**
	Building Area (m^2)	**2916.7**	**2227.6**	**2916.7**	**2850.7**	**2916.7**	**752.6**	**2916.7**	**752.6**
	Direction from Intake To Stack (°)	**75**	**217**	**12**	**240**	**82**	**176**	**51**	**176**
	Vertical Velocity (m/s)	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**
	Stack Flow (m^3/s)	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**
	Stack Radius (m)	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**
	Initial Value of σ_y (m)	**8.18**	**0**	**0**	**0**	**8.18**	**0**	**0**	**0**
	Initial Value of σ_z (m)	**9.9**	**0**	**0**	**0**	**9.9**	**0**	**0**	**0**
	Minimum Wind Speed (m/s)	**0.5**	**0.5**	**0.5**	**0.5**	**0.5**	**0.5**	**0.5**	**0.5**
	Surface Roughness Length (m)	**0.2**	**0.2**	**0.2**	**0.2**	**0.2**	**0.2**	**0.2**	**0.2**
	Sector Averaging Constant	**4.3**	**4.3**	**4.3**	**4.3**	**4.3**	**4.3**	**4.3**	**4.3**
UNIT 2	**Release Height (m)**	**29.7**	**61**	**9.8**	**7.9**	**29.7**	**61**	**9.8**	**7.9**
	Intake Height (m)	**21.2**	**21.2**	**21.2**	**7.9**	**20.4**	**20.4**	**20.4**	**20.4**
	Horizontal Distance from Intake to Stack (m)	**7.6**	**34.1**	**22.9**	**43.3**	**30.5**	**27.4**	**35.1**	**13.4**
	Elevation Difference between Stack Grade and Intake Grade (m)	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**
	Building Area (m^2)	**2916.7**	**2227.6**	**2916.7**	**2850.7**	**2916.7**	**752.6**	**2916.7**	**752.6**
	Direction from Intake To Stack (°)	**106**	**323**	**168**	**300**	**99**	**4**	**129**	**4**
	Vertical Velocity (m/s)	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**
	Stack Flow (m^3/s)	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**
	Stack Radius (m)	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**
	Initial value of σ_y (m)	**8.18**	**0**	**0**	**0**	**8.18**	**0**	**0**	**0**
	Initial value of σ_z (m)	**9.9**	**0**	**0**	**0**	**9.9**	**0**	**0**	**0**
	Minimum Wind Speed (m/s)	**0.5**	**0.5**	**0.5**	**0.5**	**0.5**	**0.5**	**0.5**	**0.5**
	Surface Roughness Length (m)	**0.2**	**0.2**	**0.2**	**0.2**	**0.2**	**0.2**	**0.2**	**0.2**
	Sector Averaging Constant	**4.3**	**4.3**	**4.3**	**4.3**	**4.3**	**4.3**	**4.3**	**4.3**

TABLE 2.3-53

ARCON96 CONTROL ROOM INTAKE χ/Q RESULTS* (sec/m^3) FOR BRAIDWOOD STATION

		Control Room Fresh Air Intake				Turbine Building Emergency Air Intake			
		Containment Wall	Plant Vent	PORVs/ Safety Valves**	Main Steam Line Break	Containment Wall	Plant Vent	PORVs/ Safety Valves**	Main Steam Line Break
UNIT 1	0-2 hour	**1.73E-03**	2.05E-03	1.57E-03	**3.17E-03**	**1.01E-03**	2.32E-03	7.66E-04	**1.67E-02**
	2-8 hour	**1.24E-03**	1.48E-03	1.30E-03	**2.73E-03**	**7.25E-04**	1.74E-03	**6.86E-04**	**1.44E-02**
	8-24 hour	**5.23E-04**	5.89E-04	5.58E-04	**1.19E-03**	**3.07E-04**	7.23E-04	**3.12E-04**	**5.99E-03**
	1-4 days	**3.55E-04**	4.44E-04	3.32E-04	**8.20E-04**	**2.07E-04**	4.57E-04	**1.80E-04**	**4.07E-03**
	4-30 days	**2.62E-04**	3.46E-04	2.14E-04	6.02E-04	**1.46E-04**	**3.62E-04**	**1.21E-04**	**2.80E-03**
UNIT 2	0-2 hour	1.64E-03	**2.22E-03**	**1.74E-03**	2.98E-03	1.00E-03	**2.46E-03**	**8.14E-04**	1.49E-02
	2-8 hour	1.15E-03	**1.80E-03**	**1.49E-03**	2.61E-03	7.10E-04	**1.87E-03**	6.84E-04	1.23E-02
	8-24 hour	4.93E-04	**7.20E-04**	**6.18E-04**	**1.19E-03**	3.01E-04	**7.41E-04**	2.70E-04	5.16E-03
	1-4 days	3.19E-04	**4.75E-04**	**3.94E-04**	7.65E-04	1.90E-04	**4.60E-04**	1.48E-04	3.06E-03
	4-30 days	2.41E-04	**3.81E-04**	**2.82E-04**	**6.60E-04**	1.39E-04	3.16E-04	1.11E-04	2.01E-03

* Bolding and shading indicates the maximum unit χ/Q value.

** PORVs/Safety Valve χ/Q values contain a factor of 5 reduction for vertical uncapped releases per RG 1.194, Section 6.

2.4 HYDROLOGIC ENGINEERING

2.4.1 Hydrologic Description

2.4.1.1 Site and Facilities

The site is located about 4 miles southwest of the Kankakee River near the town of Custer Park in a strip-mined region presently characterized by many water-filled trenches and ponds. Cooling water for the plant is supplied by a cooling pond which covers one of these strip-mined areas. (Note: the condenser water cooling facility at Braidwood Station is referred to as a cooling pond rather than as a cooling lake as in the PSAR. This is consistent with the definition of "pond" in EPA Effluent Guidelines and Standards for Steam Electric Power Generation, 40 CFR 423, Section 432.11, Item m, which became effective in 1974.) The pond has an average depth of approximately 8.21 feet at its normal pool elevation of 595 feet MSL (all elevations refer to USGS 1929 datum), with a surface area of 2475 acres or 3.87 mi^2 and a storage volume of 22,300 acre-feet at normal pool elevation. The water surface area is 73% of its total drainage area of 5.3 mi^2. The surface area of the pond at normal pool elevation, as measured from the as-built topographic maps, is 2537 acres or 3.96 mi^2. This is not significantly different from the area used for analysis. The pond is contained by dikes having a top elevation of 600 feet, except for that portion of the dike just south of the plant, which has a top elevation of 602.5 feet. The dike system is not a Seismic Category I structure.

Seismic Category I buildings include the containment building, the auxiliary building, and the fuel handling building. In addition, the portion of the lake screen house housing the essential service water (ESW) intake, the essential service cooling pond (ESCP), and the ESW intake and discharge pipes, are classified as safety Category I facilities. The grade floor elevation of these buildings is at 601.0 feet. Seismic Category I structures are shown in Figures 2.4-1, 2.4-26 through 2.4-29, and 2.4-47.

The essential service cooling pond is located in the northwestern corner of the cooling pond in an area excavated below the surrounding pond bottom, to an elevation of 584 feet. The ESCP has a surface area of 99 acres and a depth of 6.0 feet at a pool elevation of 590.0 feet (see Figure 2.4-47).

The essential service water cooling pond survey is a hydrographic survey incorporating precision water depth measurements. These measurements are performed with a survey fathometer, which produces a continuous chart recording of water depth. Slope definition as well as pond bottom survey is obtained at approximately 100-foot-wide tracklines. The initial pond survey was performed on October 21, 1981. Subsequent surveys will be performed once every 18 months;

however, this survey interval may be increased in the future. The survey program has been incorporated into the station formal inspection and monitoring program per Regulatory Guide 1.127.

Makeup water for the pond is pumped from the river screen house on the Kankakee River via pipeline to the northeast corner of the cooling pond. Blowdown water is discharged from the plant by pipeline to the blowdown outfall structure to the discharge flume or multi-port diffuser spillway for release to the Kankakee River.

The Kankakee River is joined by Horse Creek at Custer Park. Horse Creek lies about 2.5 miles east of the site at its nearest point. The Mazon River flows northwest to the Illinois River. At its closest point, the Mazon River is joined by Granary Creek, 1 mile southwest of the site and about 4 miles south of the safety-related facilities. Crane Creek, a tributary of Granary Creek, flows north to meet Granary Creek about 1.5 miles south of the site. The flow in both creeks is intermittent. Floods on these small local streams and the Kankakee River would not affect safety-related portions of the plant.

The nearest highways to the site, Illinois State Routes 53 and 129, are adjacent to the northwest boundary of the site. Interstate 55 is less than 2 miles west-northwest of the site (centerline of the reactors), and State Route 113 is approximately 2 miles north of the site. Access to the plant is via State Route 53. Onsite roads in the immediate plant area vary in elevation from 598.0 feet to 601.0 feet. The Illinois Central Gulf Railroad runs parallel to and between State Routes 53 and 129 provides spur track access to the site. Characteristics of the area surrounding the site are shown in Figure 2.4-2. Changes to existing drainage features are also shown in Figure 2.4-2.

2.4.1.2 Hydrosphere

The plant is sited on a low ridge southwest of the Kankakee River and east of the Mazon River. The Mazon River watershed map is shown on Figure 2.4-3. The Kankakee River joins the Des Plaines River about 10 miles directly north of the site to form the Illinois River at river mile 273. The Mazon River flows into the Illinois River at mile 264. Other streams in the area are Crane, Granary, and Horse Creeks, as shown on Figure 2.4-4. Except for ponded water in the strip-mined areas around Braidwood and Godley, there are no ponds in the region.

The Kankakee River rises near South Bend, Indiana, and flows southwesterly 111 miles to Aroma Park, Illinois, where it is joined by its largest tributary, the Iroquois River. The Kankakee River watershed map is shown on Figure 2.4-5. The Kankakee then flows northwesterly for 38 miles to its junction with the Des Plaines River. The Kankakee Basin is 130 miles long and a maximum of 70 miles wide. The Kankakee River drains 5280 mi^2, of which 2155 mi^2 are in Illinois and 3125 mi^2 are in Indiana. The maximum relief in the basin between the mouth and the high point on the drainage divide near Valparaiso, Indiana, is 375 feet. The drainage divide is almost entirely defined by low ridges of glacial origin. Below the state line in Illinois, the Kankakee River is 59 miles long, with widths varying from 200 to 800 feet and depths from 1 foot to 15 feet. The total fall from the state line to the mouth is 127 feet. Channel slopes vary from less than 0.5 foot per mile to over 4 feet per mile. Channel slope in the site area is about 2 feet per mile. Most of the riverbed in Illinois is on or very near bedrock. Relatively thin layers of sand and gravel overlie the bedrock, with some small areas of silt.

There are two dams on the Kankakee River, one at Wilmington, about 4 miles downstream from the intake point, and the other at Kankakee, about 15 miles upstream. The Wilmington dam is 11 feet high and forms a pool 2 miles long. The Kankakee dam is 12 feet high and forms a pool 6 miles long. Both dams are solid concrete on bedrock. Neither dam is now used for power production. Other streams in the vicinity of the plant have no control structures (Reference 1).

The flow of the Kankakee River is gauged near Wilmington about 8.8 miles downstream from the withdrawal point and 5.5 miles upstream from its mouth; the drainage area at the gauge is 5150 mi^2. The regional hydrologic network is shown on Figure 2.4-6.

The site overlies aquifers commonly used for groundwater supply. There are numerous private and public water supply wells within 10 miles of the site. Deep wells, which are usually required for municipal supply, generally are finished in the St. Peter and Galesville sandstone aquifers. Shallow wells, commonly used for individual private supplies, are driven in the sand aquifer. Specific capacities of the wells vary widely. The sandstone well water is high in mineral content and hardness. Groundwater users within 2 miles of the site are discussed in Subsection 2.4.13.2.

There is little public water use of the main streams of the site drainage area. These streams include the Kankakee River, tributaries of the Mazon River, Crane Creek, and Granary Creek. Wilmington is the only urban center of any consequence between the intake-discharge area for Braidwood on the Kankakee River (near Custer Park) and its confluence with the Des Plaines River, which forms the Illinois River at Dresden. Wilmington withdraws approximately 1 cfs from the west shoreline of the Kankakee River for its primary public water supply, four miles downstream of the Braidwood Station discharge. Wilmington's alternate water supply is from wells placed in the Ironton-Galesville sandstone, an aquifer that is not subject to local infiltration. There are no other public water supplies taken from the Kankakee, Mazon, or Illinois Rivers within 50 miles downstream from the site area.

The dam on the Kankakee River at Wilmington impounds water to a normal depth of 11 feet or less upstream for 2 miles. The water is presently used as a minor recreation center. The only industry downstream of the Braidwood Station that could be identified as a substantial user of Kankakee River water is the Joliet Arsenal, which in the past withdrew approximately 25 million gpd from the Kankakee River. However, the Joliet Arsenal has been closed since July 1977 and is no longer withdrawing water from the river. There are no plans to reopen the plant in the future. Other uses of the downstream surface water system are support of fish and other aquatic life, boating, fishing, swimming, and navigation on the Illinois River.

2.4.2 Floods

2.4.2.1 Flood History

The peak discharge, corresponding gauge height, and maximum gauge height, if higher, for each water year (October through September) of record on the Kankakee River near Wilmington are shown in Table 2.4-1. The gauge is located in the northwest quarter of Section 15, T33N, R9E, 0.4 mile downstream from Prairie Creek and about 8.78 miles downstream from the intake point. The intercepted drainage area is 5150 mi^2. The datum or zero point of the waterstage recorded is at elevation 510.86 feet above mean sea level, North American datum of 1929. Peak discharges shown for the water years 1915 through 1933 are derived from Kankakee River gauging records at Custer Park, 1/4 mile upstream from Horse Creek. The gauge intercepted a drainage area of approximately 4870 mi^2. The flow rates shown were adjusted to the Wilmington site by multiplying the Custer Park discharge by the ratio of the square roots of the drainage areas.

The maximum known discharge near Wilmington, 75,900 cfs, occurred July 13, 1957. Its corresponding gauge height was 11.40 feet above datum. The maximum stage during the period of record was 13.88 feet, caused by ice jams. Ice jam floods in 1883 and 1887 reached a stage of 16.73 feet, for which the discharge is not known. All maximum stages greater than those due to floods were caused by ice jams. Of the 36 years for which maximum water surface elevations are known, 18 maximums were caused by ice jams as high as almost 7 feet above the year's highest flood stage (see Subsection 2.4.7).

2.4.2.2 Flood Design Considerations

The plant main floor is located at elevation 601.0 feet, which is above all flood levels from nearby rivers, streams, and reservoirs. The cooling pond dike system is higher than the calculated flood elevation with coincident wind wave action. The probable maximum precipitation (PMP) water surface elevation of the pond is below safety-related facilities. Floods occurring on the Kankakee River could affect only the river screen house, which is a non-safety-related structure supplying makeup water to the cooling pond. Other streams in the area pose no flood threat to safety-related items. The site drainage system has been designed to pass rainfall without flooding.

There are no dams upstream on nearby rivers whose failure could cause flooding at the site. The general terrain of the area is flat, with no location at which landslides could cause flood waves at the site.

The controlling event for flooding at the site is the probable maximum flood for the cooling pond (see Subsection 2.4.8.2). This event has been analyzed by applying the local probable maximum precipitation (PMP) to the pond watershed following an antecedent storm equivalent to one-half the PMP (see Subsection 2.4.8.2.4).

2.4.2.3 Effects of Local Intense Precipitation

Site grading and drainage are designed to assure that the local PMP will have no effect on safety-related facilities. The layout of roads, tracks, and drainage in the immediate plant area is shown in Figure 2.4-7.

PMP data are taken from Hydrometeorological Report No. 33 (Reference 2) and is estimated to amount to 31.9 inches over a 48-hour period. This is the summer PMP, which is greater than the largest winter PMP coincident with the water equivalent of the 100-year snow pack. The PMP time distribution in 6-hour and 1-hour periods is given in Tables 2.4-2 and 2.4-3.

The roofs of all safety-related structures are designed to withstand the higher of the loads caused by the 24-hour all-season PMP or the 100-year maximum snow pack combined with the winter PMP of 48-hour duration at the plant site.

Postulating that the roof drains get clogged at the time of PMP, the maximum accumulation of water on the roofs of safety-related structures will be up to the height of the parapet walls plus the depth of overflow over the parapet wall. The height of the parapet walls is 1 foot 4 inches. The maximum depth of overflow is estimated to be 2.0 inches. Therefore, the corresponding water load due to summer PMP on the roofs will be 93.6 lb/ft^2. The maximum 48-hour winter PMP at the site is 14.7 inches in March (Reference 2). The snow load at the site corresponding to a 100-year mean recurrence interval is 28 lb/ft^2. Due to the 1-foot 4-inch high parapet walls, accumulation of the entire winter PMP with the above snow load is not possible on the roofs, and the excess precipitation overflows the parapet. Therefore, the governing roof load is 93.6 lb/ft^2. However, as explained in Subsection 3.8.4, the roofs of all safety-related structures are designed for a load of 104 lb/ft^2.

The plant grade elevation is 600.0 feet and the grade floors of the safety-related buildings are at elevation 601.0 feet.

The site drainage system is designed to follow the natural drainage pattern and to drain the storm water away from the plant area. The areas surrounding the plant buildings are graded to direct the surface runoff towards north, west, and east of the plant area.

For the analysis of local intense precipitation, the 1-hour PMP on 1 square mile area for the site is taken from the Hydrometeorological Report No. 52 (Reference 2c). This PMP was derived from 6-hour, 10-square mile PMP values given in Hydrometeorological Report No. 51 (Reference 2b), and is considered point rainfall value. This 1-hour 1-square mile PMP is distributed into values for smaller durations following procedures given in Reference 2c. The magnitude and intensity of these smaller duration rainfalls are presented in Table 2.4-4.

The runoff from local intense precipitation that can contribute to potential flooding of the plant area is from the areas shown in Figure 2.4-7a. The probable maximum precipitation falling on this area was considered in the analysis of local intense precipitation on the plant site. The peripheral roads and railroads will act as weirs to pass the resulting runoff, in the event of a PMP at the plant site. The precast concrete barriers along the outside of the security fence are considered in the analysis. These concrete barriers are 10 feet long with a 3-foot space between the adjacent barriers. Therefore, the barrier placement would cause a restricted flowpath for the PMP runoff. The barriers are placed along the edge of the existing roads or the parking areas, outside the security fence. In addition, concrete barriers 10 feet long with a maximum 2 foot spacing between the barriers are installed in areas beyond the outside security fence forming the outer vehicle barrier system. The elevation of the grade on which all the barriers are placed is considered in the analysis, and the water level upstream of the barriers is calculated assuming weir flow through the openings. The calculated water level upstream of the barriers is considered as the tail water level for the weir flow over the peripheral plant roads and railroad tracks inside the security fence to calculate the maximum water level near the plant buildings.

It was conservatively assumed in the analysis that the site drainage system would not be functioning at the time of the PMP. The rational formula was used in estimating the peak runoff from the area. The rational formula is given by:

$$Q = CIA, \text{ where}$$

Q = peak runoff (cfs),

C = coefficient of runoff,

I = intensity of rainfall (inches/hour), and

A = drainage area (acres).

The coefficient of runoff was assumed conservatively to be 1.0. The time of concentration was computed from Kirpich's formula (Reference 2a). The intensity of rainfall corresponding to a time of concentration was interpolated from Table 2.4-4. The water surface elevation was estimated for peak flow over the peripheral roads and railroad tracks and through the openings between the peripheral concrete barriers, using a broad crested weir formula with a coefficient of discharge of 2.64 (Reference 2d). Whenever the tail water level is higher than the weir crest, submergence factors are applied to the coefficient of discharge. Backwater calculations are performed for the areas from the peripheral roads and railroad tracks upstream to plant buildings to estimate the maximum water level adjacent to the buildings housing the safety-related equipment.

The plant site area is divided into inner Zones A, B, C, and D and outer Zones Ax, Bx, and Cx as shown in Figure 2.4-7a. The detailed site layout and the location of the cross sections for the backwater calculation are shown in Figure 2.4-7b.

The storm water, which accumulates in Zone A, flows through the openings in the inner peripheral concrete barriers north and west of the zone. The area inside the security fence bounded by the plant roads inside Zone A contains peripheral roads at elevation 601.0 feet, except for 180-foot length of road at elevation 600.0 feet. A calculation was performed to estimate the maximum water level in Zone A due to this constriction.

The total area of Zone A and Ax is 28.5 acres. The time of concentration is 17.6 minutes with a peak runoff from Zone A and Ax of 1012 cfs. This peak flow produces a water level of 600.8 feet upstream of the concrete barriers. The peak runoff from the inner area of 14.5 acres bounded by the plant roads is 515 cfs, and the maximum water level near the plant buildings due to this runoff over the 180-foot long road at elevation 600.0 feet is 601.31 feet.

The runoff from Zone B flows through the openings in the inner peripheral concrete barriers north, east, and south of the zone. The runoff from the area bounded by railroad tracks and plant roads inside the security fence in Zone B flows over railroad Track 1 on the east, over railroad Track 3 on the south, and the plant road on the north.

The area of Zone B and Bx is 60.85 acres. With a concentration time of 26.8 minutes, the peak runoff from Zone B and Bx is 1675 cfs. This peak flow produces a water level of 601.5 feet upstream of the outer vehicle barrier system. The peak runoff from the area of 22.3 acres inside Zone B, bounded by the railroad tracks and plant roads, is 625 cfs. The maximum water level near the plant buildings is 601.85 feet due to peak flow over 880 feet of track at elevation 601.0 feet, including the effect of backwater.

Zone C has an area of 8.99 acres. The runoff from this zone flows over the peripheral road on the north and west and the railroad track on the east. The time of concentration for this area is 12.6 minutes, and the peak runoff is 369 cfs. The peak runoff flows over the peripheral road and 2,000 feet of track at an elevation of 601.0 feet, and produces a maximum water level in Zone C of 601.17 feet.

The south area of Zone C and Zone Cx has an area of 23.4 acres. The runoff from this area flows northwest over the outer vehicle barrier system, northwest of Zone Cx. The time of concentration for this area is 19 minutes, and the peak runoff is 1622.5 cfs. The peak runoff produces a maximum water level of 601.05 feet. The backwater computation for Zone Cx determined the maximum water level in Zone C of 601.43 feet which is the governing maximum water level in Zone C.

The runoff from Zone D flows over the peripheral roads toward the south, east, and west. The drainage area of Zone D is 10.85 acres, and the peak runoff is estimated to be 413 cfs, with a time of concentration of 16.5 minutes. This peak flow over 1320 feet of the peripheral road at elevation 601.0 feet produces a maximum water level of 601.55 feet near the north boundary of Zone D.

The security fence around the plant buildings has the top of concrete footing at the grade elevation and, hence, will not affect the water levels estimated in the foregoing analysis.

It can be seen from the foregoing analysis that the water level in the area adjacent to the plant buildings can reach a maximum elevation of 601.85 feet, which is 0.85 foot above the grade floor level. This maximum elevation is less than the top of the 15-inch (maximum) steel barriers that are installed at the openings to buildings housing safety-related equipment. Therefore, the safety related equipment of Braidwood Station is not affected by local probable maximum precipitation. The elevation at the top of the steel barriers is 602.25 feet.

This occurs for a short period of time until the runoff due to the PMP is discharged from the plant site over the roads and railroads and the drainage system. To prevent this water from entering areas where essential equipment/systems are located, reinforced concrete curbs or steel barriers are provided at the following locations, at a minimum:

a. External hatches to the RWST tunnel
b. Radwaste building access to the transfer tunnel
c. MSIV rooms adjacent to 401’ elevation exterior access doors
d. Personnel access locations to the Auxiliary Building from the FHB 401’ elevation

2.4.3 Probable Maximum Floods (PMF) on Streams and Rivers

The probable maximum flood (PMF) is defined by the U.S. Army Corps of Engineers as the hypothetical flood characteristics that are considered to be the most severe reasonably possible at a particular location, based on relatively comprehensive hydrometeorological analysis of critical runoff-producing precipitation and hydrologic factors favorable for maximum flood runoff.

PMF elevations were calculated for the Kankakee River, Mazon River, Granary Creek downstream from Crane Creek, and the cooling pond. The Kankakee River PMF elevation at the river screen house and elevations for other streams near the site are calculated and described in the following paragraphs. The cooling pond PMF is evaluated in Subsection 2.4.8.2. These locations and water levels are summarized in Table 2.4-5 along with the low, average annual, and flood-of-record flow elevations for the Kankakee River at the intake. As shown in the following subsections, none of the stream floods would have any effect on the plant safety-related systems.

The unit hydrograph and PMF hydrograph for the Kankakee River at the Wilmington gauge are given in Figure 2.4-8. The PMF hydrograph was constructed by applying the PMP for the area (Reference 2)(after infiltration losses) to the synthetic unit hydrograph determined by the method developed by the State of Illinois Division of Waterways for Illinois streams (Reference 3).

The PMF hydrograph for the Kankakee River at the Wilmington gauge yields a peak discharge of 211,981 cfs. This flow was adjusted to the intake location by multiplying the peak discharge at the Wilmington gauge by the ratio of the square roots of the drainage areas. This results in a PMF peak discharge of 209,000 cfs at the intake. The corresponding flood elevation is 561.3 feet above mean sea level. This elevation was determined by relating flow as measured at Wilmington, Illinois, by the U.S. Geological Survey to the corresponding stages measured above Custer Park, Illinois (close to the intake point), by the Army Corps of Engineers, as shown in Figure 2.4-9. The rating curve for the Kankakee River at the Wilmington gauge as developed by the U.S. Geological Survey from actual measurements is shown in Figure 2.4-10.

The plant grade elevation of 600 feet is 38.7 feet higher than the estimated PMF elevation. The PMF line would be 3 miles away from the site facilities. Hence, the PMF on the Kankakee River would pose no threat to the site facilities.

2.4.3.1 Probable Maximum Precipitation (PMP) on the Kankakee River, the Mazon River, and Crane and Granary Creeks

The probable maximum precipitation (PMP) was derived from Hydrometeorological Report No. 33 (HMR-33). The PMP estimates in HMR-33 involve detailed analyses of actual flood-producing storms and modification and extrapolation of historic data to reflect the most severe rainfall possible.

The greatest PMP would most likely occur in July or August; slightly smaller amounts of precipitation are possible in June or September. Spring PMP occurring during a period of high snow melt runoff would cause a flood substantially smaller than the summer PMP. This is because the all-season PMP is considerably greater than the sum of spring PMP and the water equivalent of a 100-year snow cover (Reference 4). The point 48-hour all season PMP is 31.9 inches, (Reference 2) while the sum of corresponding April PMP and the water equivalent of 100-year snow cover is only 21 inches.

The PMP for the Kankakee River, the Mazon River, and Crane and Granary Creeks was taken from HMR-33. The PMP was not reduced in consideration of the storm's isohyetal orientation to the basin shape. The time distribution of the PMP for Crane and Granary Creeks, the Mazon River, and the Kankakee River is given in Table 2.4-6. Since the unit hydrograph rainfall duration is 3 hours for Crane and Granary Creeks and the Mazon River, the precipitation increments are given for 3-hour intervals. For the Kankakee River, the PMP was distributed into 6-hour intervals.

2.4.3.2 Precipitation Losses on the Kankakee River, the Mazon River, and Crane and Granary Creeks

Part of a storm's rainfall is retained by the basin and does not contribute to storm runoff. The significant factors in basin retention are depression storage, interception by vegetation, and soil infiltration. The rate of retention depends largely on soil types, land use, and antecedent soil moisture.

For the Kankakee River Basin, initial retention was assumed to be 0.5 inch, with infiltration thereafter taken to be 0.1 inch per hour. For the Mazon River and Crane and Granary Creeks, initial retention was assumed to be 1.0 inch, with subsequent infiltration taken as 0.1 inch per hour. Selection of these conservatively low values was based on land use and the soils in the basins. Soils (References 1 and 5) in the Mazon River Basin in the area are evenly mixed between hydrologic class C (minimum infiltration 0.05 to 0.15 in./hr) and B (0.15 to 0.30 in./hr) (References 6 and 7).

2.4.3.3 Runoff and Stream Course Models for the Kankakee River, the Mazon River, and Crane and Granary Creeks

The basins are shown in Figures 2.4-3, 2.4-4, and 2.4-5.

The unit hydrograph and PMF hydrograph for the Kankakee River at the Wilmington gauge are given in Figure 2.4-8. The PMF hydrograph was constructed by applying the PMP for the area (Reference 2) (after infiltration losses) to the synthetic unit hydrograph determined by the method developed by the State of Illinois Division of Waterways for Illinois Streams (Reference 3).

Reference 3 provides a generalized method of computing flood hydrographs based on an extensive study and verification of several Illinois streams. In view of the insignificance of the river PMF levels for plant safety, it was found reasonable to utilize the previously established watershed parameters.

Unit hydrographs for the Mazon River at section X-MD2 below the East Fork Mazon River and the combined Crane-Granary Creeks basin are shown on Figures 2.4-11 and 2.4-12. They were computed using the Snyder synthetic method. Some important parameters for the basin characteristics and the unit graphs are given in Table 2.4-7. The values of 640 C_p and C_t for the Mazon River at Coal City (drainage area 470 mi^2) are 530 and 3.7 respectively (Reference 3). The 640 C_p value of 530 is considered to be conservatively high for the basin topography. C_t and C_p are coefficients in Snyder's synthetic-unit hydrograph formulas and depend on basin characteristics. For the combined Crane and Granary Creeks basin, a higher C_t of 5.0 and a lower 640 C_p of 320 were used. Although these coefficients yield low peaks, they are considered to reasonably represent flood-producing characteristics of this watershed, since the basin is long and narrow and there are strip-mined areas in the downstream portion of the basin which will retard flood runoff from the basin.

2.4.3.4 Probable Maximum Flood Flow on the Kankakee River, the Mazon River, and Crane and Granary Creeks

The PMF hydrograph (Figure 2.4-8) for the Kankakee River at the Wilmington gauge yields a peak discharge of 211,981 cfs. This flow was adjusted to the intake location by multiplying the peak discharge at the Wilmington gauge by the ratio of the square roots of the drainage areas. The drainage area of the Kankakee River at the Wilmington gauge is 5,150 mi^2. The drainage area at the intake is 5,000 mi^2. The drainage area at the intake is 5,000 mi^2. This results in a PMF peak discharge of 209,000 cfs at the intake.

The PMF hydrographs for the Mazon River and for Crane and Granary Creeks are shown in Figures 2.4-13 and 2.4-14 respectively. The hydrographs include a base flow equal to the lowest mean daily flow in the wettest month of record on the Mazon River at Coal City transposed to the flood site by the ratio of the drainage areas. The base flow on the Mazon River is 200 cfs and 50 cfs for Crane and Granary Creeks. The PMF peak discharge on Granary Creek at a point about 1 mile south-southwest of the southwest corner of the site, just upstream from its junction with the East Fork Mazon River, is 19,500 cfs. The peak discharge on the Mazon River at old Route 66 at section X-MD2 is 112,000 cfs. There are no dams on any of the streams to affect the PMF flow. The dam at Kankakee on the Kankakee River will not affect its PMF flow. No channel routings were made.

2.4.3.5 Water Level Determination for the Kankakee River, the Mazon River, and Crane and Granary Creeks

The daily flows for the Kankakee River at Wilmington, Illinois for the period 1950 to 1970 were obtained from the data published by the U.S. Geological Survey, and the daily stage data for the Kankakee River at Custer Park, Illinois was obtained for the same period from the U.S. Army Corps of Engineers. From this data, the observed peak river flows at Wilmington for each year and the corresponding peak river elevations at Custer Park were obtained and plotted in Figure 2.4-9. The discharges at the river screen house are estimated from the discharges at Wilmington in the ratio of the square root of corresponding drainage areas. The water levels for a given discharge at the river screen house are conservatively assumed to be the same at Custer Park, which is approximately 1/2 mile upstream of the river screen house.

The peak discharge of record at Wilmington, Illinois is 75,900 cfs with a corresponding peak stage of 551.5 feet at Custer Park, Illinois and 522.26 feet at Wilmington, Illinois. The distance between the two gauging stations is 9.4 miles. Therefore, the water surface slope of the Kankakee River for a flow of 75,900 cfs is computed to be 0.000589. Using this water surface slope and the typical cross section of the river shown in figure 2.4-14a, the Manning's "n" value is computed to be 0.03, corresponding to the flow of 75,900 cfs. The water surface elevations in the river cross section were obtained for higher flows of 209,000 cfs and 400,000 cfs using the above values of Manning's "n" and the water surface slope, and the discharge rating curve is extrapolated as shown in Figure 2.4-9.

The water level in the Kankakee River near the river screen house is 561.3 feet for the estimated PMF peak discharge of 209,000 cfs. This elevation was determined by relating flow as measured by the U.S. Geological Survey at Wilmington, Illinois to the corresponding stages measured above Custer Park, Illinois (close to the intake point) by the U.S. Army Corps of Engineers, as shown in Figure 2.4-9. The rating curve for the Kankakee River at the Wilmington gauge as developed by the U.S.

Geological Survey from actual measurements is shown in Figure 2.4-10. However, if the peak discharge is conservatively postulated to be 400,000 cfs, the estimated peak water level at the screen house would be approximately 571.0 feet. Since this peak elevation is 29 feet below the plant grade elevation of 600 feet and the plant site is 4 miles away from the Kankakee River, the plant site will not be affected by the PMF in the Kankakee River.

The peak PMF discharges for the Mazon River and Granary Creek are presented at the locations where the design basis water level is determined. The Mazon River PMF peak discharge is 112,000 cfs at cross section X-MD2 with a corresponding water level of 581.5 feet as shown in Table 2.4-8. The PMF peak discharge in Granary Creek just upstream of its confluence with East Fork Mazon River is 19,500 cfs and the corresponding water level is 576.0 feet. The energy gradients at cross section X-MD2 on the Mazon River and at the cross section of Granary Creek just upstream of its confluence with East Fork Mazon River are 0.000345 and 0.000746, respectively. The cross section of Granary Creek just upstream of its confluence with East Fork Mazon River is presented in Figure 2.4-17a.

The plant grade elevation of 600 feet is 38.7 feet higher than the estimated PMF elevation. The PMF line would be 3 miles away from the site facilities. Hence, the PMF on the Kankakee River would pose no threat to site facilities.

Peak discharge elevations for the Mazon River and Crane and Granary Creeks, as listed in Table 2.4-5, were approximated by assuming normal depth of flow through a cross section at the points of interest, just upstream from old Alternate Route 66 on the Mazon River (Figure 2.4-18) and on Granary Creek just upstream of East Fork Mazon River (Figure 2.4-17a). An overall Manning's "n" of 0.06 was used based on field observations correlated with known values (Reference 8). Because the peak elevations are 18 feet or more below the plant safety-related facilities grade of 600 feet, refinement of "n" values and determination of the elevations in more detail by backwater computations is not warranted. An increase of 0.02 in "n" value adds approximately 1 foot to the water surface elevation.

Two typical cross sections for each of the streams, the East Fork Mazon River, Crane and Granary Creeks, and the Mazon River below Crane and Granary Creek are shown on Figures 2.4-15, 2.4-16, 2.4-17, and 2.4-18. The locations of these cross sections are shown in Figure 2.4-19. The rating curves at these cross sections are shown on Figures 2.4-20, 2.4-21, 2.4-22, and 2.4-23. The flood elevations in the site vicinity are tabulated in Table 2.4-8.

2.4.3.6 Coincident Wind Wave Activity

The intake structure on the Kankakee River is not a Seismic Category I structure and is designed for the flood of record only. During the PMF at elevation 561.3 feet, the wind velocity of 40 mph will produce a significant wave height of 0.8 foot. During the flood of record, elevation 552.0 feet, a wind velocity of 40 mph will produce a significant wave height of 0.65 foot. The intake structure operating floor (elevation 557 feet) is located above the flood of record, elevation 552 feet, plus wind wave (0.8). Coincident wind wave activity on other rivers and creeks which would amount to 1 or 2 feet at most will not affect the plant safety-related facilities.

2.4.4 Potential Dam Failures, Seismically Induced

The nearest upstream dam on the Kankakee River is at Kankakee, about 15 miles from the river screen house. The dam is 12 feet high, with a normal pool elevation of 595 feet. Failure of the dam would create minor flood waves which would dissipate before reaching the site area. During river floods, the dam would be completely submerged, so that failure would not cause a flood wave.

The nearest downstream dam is at Wilmington, approximately 5 miles from the river screen house; the dam is 11 feet high, with a crest elevation of 530.5 feet. A rock ledge across the river 7700 feet upstream of the dam maintains a pool elevation of 534 feet during low flows. Thus, failure of this dam due to flood flows or seismic disturbance would in no way affect safety-related portions of the plant.

The maximum water level in the pond during its PMF would be 598.17 feet; the grade of safety-related facilities is 600.0 feet. Thus, failure of the pond dikes would not affect safety-related facilities.

2.4.5 Probable Maximum Surge and Seiche Flooding

Surge and seiche flooding are not possible because there is no large body of water near the site.

2.4.6 Probable Maximum Tsunami Flooding

Tsunami flooding is not possible because the site is not near a coastal area.

2.4.7 Ice Effects

Ice flooding, which is common on the Kankakee River, could affect only the river screen house. In 17 of the most recent 34 years of record at the Wilmington gauging station, the highest yearly water levels were caused by ice jams. At such times, ice forms all along the Kankakee River in Illinois.

Major ice jams (such as those which occurred in 1866, 1883, and 1887) caused stages much higher than have been observed for flood discharge alone but would be lower than the PMF level. According to the Woermann profile of 1927, the 1866 ice jam caused a stage of 553.0 near Horse Creek. The 1883 ice jam destroyed the railroad bridge at Custer Park and displaced the approach embankments several feet downstream; it then completely destroyed the upper dam at Wilmington. Just before failure of the dam, the jam was reported to be 20 feet higher than the crest elevation of 545.0 (the present crest is at 530.5) (Reference 9). The maximum elevation upstream from Custer Park was 554.5, which occurred on February 15, 1959, due to an ice jam. Ice flooding is therefore expected to raise the water surface near the intake to a maximum elevation of 555.

Ice and ice flooding on the tributaries outside the cooling pond will not affect the plant facilities. The major tributary closest to the plant is the East Fork Mazon River, which lies about 1 mile southwest of the site at its closest point. Because of this distance from the site and the wide floodplain of the river, there will be no adverse effects on safety-related facilities due to ice in the river and subsequent flooding.

2.4.8 Cooling Water Canals and Reservoirs

2.4.8.1 Pipelines

Makeup from the Kankakee River is pumped by a 48-inch underground pipeline uphill to the cooling pond. The maximum gross withdrawal rate is 160 cfs for two units. Blowdown is discharged by a parallel 48-inch pipeline back into the Kankakee River at a maximum rate of 55.7 cfs during normal operation (see Subsection 2.4.11.5). The National Pollutant Discharge Elimination System (NPDES) permit allows for a maximum 66.8 cfs average blowdown rate over a 24-hour period.

2.4.8.2 Cooling Pond

Figure 2.4-24 shows the overall plan of Braidwood Pond. The pond has a normal pool elevation of 595 feet with a surface area of 3.87 mi^2. The total drainage area of the pond is 5.3 mi^2. The normal volume of the cooling pond is about 22,300 acre-feet. The pond and dike system is designed to withstand the probable maximum flood with coincident wind waves. The cooling pond reservoir storage-elevation curves are shown in Figure 2.4-25. During pond construction, excavated material was placed in the deep trenches and on islands in the pond. Figures 2.4-26, 2.4-27, 2.4-28, and 2.4-29 show pond excavation and fill.

2.4.8.2.1 Probable Maximum Precipitation on the Pond

The probable maximum precipitation (PMP) was derived from Weather Bureau Hydrometeorological Report No. 33 (Reference 2). The greatest PMP would most likely occur in July or August; slightly smaller amounts are possible in June or

September. Spring PMP occurring during a period of high snow melt runoff would cause a flood substantially smaller than the summer PMP on the pond basin. The resulting PMPs for various duration storms are given in Table 2.4-2.

2.4.8.2.2 Precipitation Losses

A small part of the PMP is retained by the basin and does not contribute to storm runoff. A large part (73%) of the pond drainage basin is covered by the pond. The remaining area is a strip-mined area and consists of the perimeter dike, the interior dikes, and islands. Many ponds at different elevations exist in the area, indicating poor drainage and impervious soils. These characteristics indicate that initial basin land retention will be high and subsequent infiltration will be very low. Consequently, initial retention of rainfall was assumed to be 1.0 inch and infiltration 0.05 inch per hour. Both values are considered to be conservatively low, resulting in a conservatively high runoff. An infiltration rate of 0.05 inch per hour falls within the range for hydrologic soil Group D, the soils with the highest runoff potential. Actual soils in the basin probably are in a class with lower runoff potential.

The initial loss of 1.0 inch was applied only to that part of the drainage basin excluding the pond surface area. No initial loss was considered for the pond area, which is 73% of the total drainage area. Due to the presence of strip-mined area, poor drainage conditions, and impervious soil, an initial loss of 1.0 inch was used for the drainage basin excluding the pond area. This amounts to an initial loss of 0.25 inch over the total drainage area. However, if a more conservative loss of only 0.125 inch over the total drainage area is assumed, it will amount to an additional volume of water of 35 acre-feet in the cooling pond from the 5.3 mi^2 drainage area. This additional amount of water will raise the maximum PMF water level by 0.014 feet, which will not have any effect on the safety-related facilities of the plant.

2.4.8.2.3 Runoff Model

Since the pond surface area occupies a very large portion of its drainage area, all land runoff was considered to enter the pond instantaneously. Since instantaneous response of the area draining into the pond is assumed, the resulting inflow rate and pond elevations would be conservatively higher than those resulting from a unit hydrograph approach. Actually, ground depressions due to strip mining would add to the overland flow time lag between rainfall and runoff into the pond, resulting in flow rates lower than those computed.

Rain falling onto the pond would have no retention losses. Rain falling onto the ground was reduced by basin retention to determine surface runoff. Thus, total runoff volume was the total rainfall volume on the total drainage basin minus initial retention and infiltration on the basin area excluding the pond surface area. The resulting PMF runoff volume and peak inflow rate are given in Table 2.4-9.

2.4.8.2.4 Probable Maximum Flood Flow for Cooling Pond

The initial cooling pond elevation for the PMF routing is estimated to be 596.10 feet. This is based on an antecedent storm equivalent to one-half the maximum 6-hour PMP occurring 3 days prior to the PMP. Given the area-capacity curve for the cooling pond and based on outflow over the 200-foot-wide broad-crested spillway (Figures 2.4-30 and 2.4-31), the initial elevation was estimated by using the U.S. Army Corps of Engineers program for spillway rating and flood routing (Reference 10). The elevation 596.10 feet would occur at the end of the third day of the storm.

Superimposing the 48-hour PMP distribution as given in Table 2.4-3 on the above water surface elevation of 596.10 feet results in the maximum water surface elevation of 598.17 feet MSL.

The PMF inflow hydrograph is shown on Figure 2.4-32. The spillway crest length of 200 feet gives a maximum reservoir rise of 3.17 feet above normal pool. Peak discharge under this condition would be 2184 cfs. The PMF outflow hydrograph is also shown on Figure 2.4-32. The spillway rating table is shown as Table 2.4-10. Wind wave action is discussed in Subsection 2.4.8.2.6.

2.4.8.2.5 Water Level Determinations

The time history of the pond water surface elevation during the PMF is shown on Figure 2.4-33. It would take about 8 days for the PMF surcharge above the spillway crest to be evacuated.

2.4.8.2.6 Coincident Wind Wave Activity

Wind wave action, wind tide, and runup were considered at various locations around the cooling pond. Wave determinations were made for various fetches, wind speeds, and pool elevations. Pond depth was computed as the average depths at regular intervals along the wind path, giving more weight to depths closer to the runup area. Setup and runup were computed by methods presented in References 11, 12, and 13. Shallow-water significant wave height, wave period, wave length, and equivalent deep-water wave height and length were computed from information in Reference 13.

The potential of flooding and coincident wind wave activity has been investigated at several locations, as shown in Figure 2.4-34. The cooling pond dike is a non-Seismic Category I structure. Coincidental PMF without antecedent Standard Project Flood (SPF) and a 25-mph overland wind were taken as the design basis for establishing the top elevation of the outer dike system except for the dike at location A. Runup for the largest open area at location C was computed for dumped riprap for a 1:3 slope. The wave runup including setup was calculated to be 1.80 feet above the PMF pool level of 597.91 feet. The wave runup calculations were made based on the shallow-water wave theory (References 11 and 13) because of shallow depth consideration. Similar wind wave studies were also made for the dikes at locations B, D, and E. The deep-water wave theory (References 11 and 12) was used at location D because of the increased depth and reduced wave length. Results of this analysis are given in Tables 2.4-11 and 2.4-12. Based on these results, the top elevation of the outer dike system except for location A is 600.0 feet to eliminate the potential for wave overtopping under this event.

The integrity and stability of the exterior dike A to the north of the essential cooling pond (Figure 2.4-34) were analyzed by considering the maximum wave due to 40-mph wind on PMF pool with an antecedent SPF condition. It was assumed that the interior dike south of the essential cooling pond would not exist. The largest fetch resulting from this assumption is at location A. The wave runups (including setups) were calculated based on a shallow water condition and are 3.10 feet and 4.17 feet for the significant and maximum waves, respectively. Superimposing the wave runup values on the PMF level of 598.17 feet resulted in a wave runup elevation of 601.27 feet for significant waves and elevation 602.34 feet for maximum waves at location A. In order to provide protection for this extreme event, the exterior dike at location A was built 2.5 feet higher than at the other locations. The top elevation of this dike is 602.5 feet.

The protection of the pondside slope of the dikes against wind wave action is based on the local wind wave characteristics. The basis used to determine the required riprap sizes and thickness of the riprap layer is extreme wind (60 mph) over normal pool or 25-mph wind (40 mph at location A) over PMF pool, whichever produces the maximum wave height. Based on these design conditions, the riprap sizes and thicknesses were determined using the procedures defined in Reference 13. For instance, the significant wave height generated by the 40-mph wind over PMF pool (598.17 feet) governs the design criteria for the portion of dike at location A (Figure 2.4-34). The 40-mph wind velocity will produce a significant wave height of 2.35 feet (Table 2.4-11). An 18-inch thick riprap with at least an average stone weight of 68 pounds (maximum weight of 250 pounds and minimum weight of 5 pounds) laid on a 12-inch thick gravel bedding is provided. Details of the riprap are shown in Figure 2.4-35.

The effects of wave action from the most severe winds which can reasonably be postulated for the site, noncoincident with a severe flood, were evaluated. The most severe wave action would not affect any safety-related structure.

The wind speed used was the highest speed over water with sufficient duration to develop waves fully. A curve of speed versus duration for a 100-year wind over land was developed using the fastest-mile speed and limited data for 5-minute and 1-hour speeds. The 100-year fastest-mile speed over land for the site is 85 mph (Reference 14). Both 5-minute and 1-hour speeds were computed from Corps of Engineers data for midwest and southwest states (Reference 15). The 100-year curve was drawn through the fastest-mile point and the derived 5-minute and 1-hour points.

For the site, the 100-year wind over land with sufficient duration (about 19 minutes) to develop waves fully is about 60 mph. The corresponding speed over the reservoir is about 70 mph. The fetches and embankment conditions are the same as described in Subsection 2.4.8.2.6 for 25 mph. The effective average depth at normal pool is about 8.21 feet below the normal pool elevation of 595.0 feet. This is 3.17 feet lower than for PMF pool. Wind wave characteristics and heights above normal pool for the extreme wind are shown in Tables 2.4-13 and 2.4-14 respectively.

The maximum wind wave runup and forces caused by a sustained wind of 40 mph over land coincident with PMF pool and an antecedent SPF were considered in the design of the pond screen house located at the north end of the essential cooling pond. The bottom elevation of the essential service cooling pond (ESCP) is at elevation 584 feet 0 inch. Locally, the grade around the lake screen house on the lake side is at elevation 570 feet 2 inches. Due to the large depth available in comparison to the wave height (Tables 2.4-11 and 2.4-13) at the lake screen house, the breaking waves will not occur for different lake levels; however, nonbreaking waves will occur. An average water depth of 11.5 feet was used in the wind wave analysis. The height of the maximum wave would be 3.92 feet, and the maximum wave runup elevation would 602.34 feet. The forces due to nonbreaking waves were computed based on the procedures given in Reference 13. These forces are used as the design bases for the pond screen house. The forces due to wind generated waves and relevant wave parameters are presented in Table 2.4-27.

2.4.9 Channel Diversions

There is no historical or topographical evidence indicating that flow in the Kankakee River can be diverted away from its present course. The river is wide, and there are no deeply incised gorges upstream where landslides could entirely cut off river flow. Upstream ice jams will not divert flow completely, since they do not prevent overbank or subsurface flow.

There have been numerous ice jams on the Kankakee River. The lowest winter-spring flow at the Wilmington gauge during the period of record 1961-1975 was 318 cfs, which is 54% higher than the lowest recorded flow of 204 cfs. In the unlikely case that upstream icing causes flow at the river screen house to drop below the required operation level, the cooling pond will temporarily operate under closed cycle.

2.4.10 Flooding Protection Requirements

The PMF design bases for the flood protection requirements on the Kankakee and Mazon Rivers and Crane and Granary Creeks are discussed in Subsection 2.4.3. The cooling pond PMF design base is explained in Subsection 2.4.8.2. The design basis for flooding effects due to the local PMP is discussed in Subsection 2.4.2. The flood elevations resulting from any of these extreme events would have no effect on safety-related facilities.

Floods on the Kankakee River could affect the river screen house, a non-safety-related structure. The PMF on the Kankakee River could not affect the site, since the maximum water surface elevation would be a minimum of 38 feet below the plant grade elevation of 600 feet. Maximum water surface elevations under PMF conditions for the Mazon River and Granary Creek are 582 feet and 576 feet respectively, still well below the plant grade.

Floods resulting from the local PMP could result in a short-term maximum water surface elevation above the grade floor level of 601.0 feet in the immediate plant area. As described in Subsection 2.4.2.3, the essential equipment/systems are protected from the local PMP by the provision of reinforced concrete curbs or steel barriers.

The maximum water surface elevation in the cooling pond resulting from the PMF with antecedent SPF is 598.17 feet. The cooling pond dike elevation is 600 feet, except for that area south of the plant where the dike elevation is 602.5 feet. The dike is maintained at a higher elevation in this area to prevent splashover resulting from possible runup to elevation 602.34 feet (Subsection 2.4.8.2.6).

The slopes of the dikes are protected with riprap designed using procedures defined in Reference 13. The pondside slopes are designed to withstand conditions resulting from the PMF with antecedent SPF and 40-mph wind as discussed in Subsection 2.4.8.2.6. Downstream slopes are protected against erosion by seeding. The slope stability analysis for the dikes is presented in Subsection 2.5.6.

2.4.11 Low Water Considerations

2.4.11.1 Low Flow in Rivers

Low flows in the Kankakee River cannot affect safety-related facilities of the plant, since the ultimate heat sink is independent of flows in the river. Low flow elevations in the Kankakee River at the site are controlled by a rock ledge across the river between Resthaven and Lakewood Shores, 7700 feet upstream of the Wilmington dam. The ledge acts like a dam by creating a pool of water that reaches upstream to Custer Park, approximately 1 mile upstream from the intake. Under low flow conditions, the rock ledge, which is at elevation 534 feet, maintains a minimum water elevation of 534 feet.

Average flow elevations are estimated based on water level data at Custer Park and discharge data at Wilmington. These are shown in Table 2.4-15 and plotted in Figure 2.4-9. For the historical average flow of 4071 cfs at Wilmington, the stage is 538 feet. Thus, the stage at the intake, less than 0.5 mile downstream from Custer Park, is taken to be 538 feet for the average flow of 3952 cfs.

2.4.11.2 Low Water Resulting from Surges, Seiches, or Tsunami

Surges, seiches, or tsunami are not possible since the site is not near a large body of water.

2.4.11.3 Historical Low Flow

Low flow rates and frequencies for the Kankakee River at the river screen house, as shown in Table 2.4-16, were derived from the Wilmington gauge statistical data (Reference 16). The drainage area at the intake is 5000 mi^2. The drainage area at the Wilmington gauge is 5150 mi^2. Based on the ratio of the drainage areas, the low flows at the river screen house are calculated to be 97% of the Wilmington low flows.

The lowest daily flow at the Wilmington gauge for the period of record was 204 cfs on August 1, 1936. Since there is no gauge at the site area, historical low flow elevations are unknown. However, the rock ledge downstream controls low water levels and will maintain a water elevation of at least 534 feet at the River Screen House under all low flow conditions.

2.4.11.4 Future Controls

Future uses of Kankakee River water are not expected to lower minimum flows significantly. The urban Kankakee area can be expected to make a gradual increase in its withdrawal rate for public and industrial water supply, but most of that supply returns to the river as wastewater. It is possible that the city of Joliet may utilize the Kankakee River to supplement its water supply in the future (Reference 1). However, it is likely that the withdrawal point would be downstream of the plant intake. At the present time, historical data indicate that low flows have not shown a decreasing trend, since the lowest recorded flow at the Wilmington gauge occurred 39 years ago.

2.4.11.5 Plant Requirements

The required minimum cooling water flow is 89 cfs. The source of this water is the essential service cooling pond (ESCP), which is excavated within the cooling pond. Water drawn from the ESCP is unaffected by low flows in the Kankakee River. The essential service water intake at the pond screen house is at elevation 572.67 feet. The ESCP constitutes a closed cycle system as described in Subsection 9.2.1.2 and has a capacity adequate for a 30-day cooling water supply.

Makeup water for the cooling pond is withdrawn from the Kankakee River. The anticipated maximum gross withdrawal rate is 160 cfs. The average makeup withdrawal from the Kankakee River at 100% load factor is 119.8 cfs, out of which 55.7 cfs is returned to the river as blowdown. Therefore, the net makeup withdrawal from the river is 64.1 cfs. It is estimated that 66 cfs is consumed due to seepage and evaporation during maximum makeup. Actual consumptive use may be less than 66 cfs, depending on the plant operating level and seasonal variability of evaporation and blowdown. The sump invert in the river screen house at elevation 526 feet is well below the estimated minimum river water surface elevation of 534 feet.

The Licensee has agreed to the following conditions set by the Illinois Department of Conservation on water withdrawal from the Kankakee River:

a. To limit withdrawal of Kankakee River water to a maximum of 160 cfs.

b. To stop withdrawing water from the Kankakee River when the flow in the river is 442 cfs (7-day, 10-year low flow) or less, and not to withdraw water such that the flow of the river is diminished below 442 cfs.

The low flow frequency and duration information for the Kankakee River at the river screen house is shown in Table 2.4-16. During short periods of time when the net makeup water of 64.1 cfs is not available from the river, the cooling pond water levels will draw down temporarily. This drawdown does not affect the normal plant operability.

Blowdown from the plant cooling pond is discharged to the Kankakee River. The thermal mixing zone is consistent with regulations specified by the Illinois Pollution Control Board.

2.4.11.6 Heat Sink Dependability Requirements

The normal source of cooling water for the plant is the 2537-acre cooling pond. Cooling water is taken from the pond at the Pond Screen House by six circulating water pumps. Two 192-inch circulating water pipes carry water to the plant and back again to the pond. A buried pipeline from the plant takes blowdown to the Kankakee River. Makeup water is pumped from the river screen house on the Kankakee River through a buried pipeline to the northeast section of the cooling pond. Should makeup water be eliminated by system failure or extreme low flows, the pond can operate under a closed cycle system. Emergency shutdown water is available from the ultimate heat sink, namely the ESCP.

The ESCP is an excavated area located within the cooling pond designed to provide sufficient volume to permit plant operation for a minimum 30-day period without requiring makeup water in accordance with Regulatory Guide 1.27. The ESCP has been reviewed to determine its ability to handle the total heat dissipation requirements of the station assuming a LOCA coincident with a loss of offsite power on one unit and the concurrent orderly shutdown and cooldown from maximum power to cold shutdown of the other unit using normal shutdown operating procedures, a single active failure, and a coincident design basis seismic event. It is estimated that water loss due to seepage and evaporation would amount to a 1.5 foot (1 foot due to evaporation and 0.5 foot due to seepage) decrease in depth of water in ESCP for such a 30-day period (see Subsection 9.2.5). The ESCP has an area of 99 acres and a depth of 6.0 feet at elevation 590.0 feet. Its area-capacity curve is given in Figure 2.4-46. Figures 2.4-47 and 2.4-48 show the ESCP and its sections and pipelines.

The intake pipes for the essential service water are in the pond screen house at a centerline elevation of 572.67 feet, over 11 feet below the bottom of the pond. The sump invert elevation of the pond screen house is 570.17 feet. At a minimum ESCP elevation of 573.92 feet at which the 30-inch intake pipes are fully submerged, the essential service water pump net positive suction head requirements are more than satisfied. This is based upon two pumps being supplied with water at their rated pumping capacity from a single 48-inch supply line and three 30-inch intake lines. Plan and elevation drawings of the pond screen house are provided in Drawing M-19. The intakes are protected from ice blockage by traveling screens, bar grills, and trash rakes, located at the front of the Pond Screen House. The minimum operating level is 590 feet, at which point the ESCP loses communication with the cooling pond. The essential service water pumps are located in the auxiliary building. Two essential service water discharge pipelines run from the auxiliary building to the south end of the ESCP. The description of the essential service water system can be found in Subsection 9.2.1.2.

The safety-related components of the pond screen house and essential service water structure have been designed to withstand the dynamic wave effects resulting from a 60-mph wind acting over a normal pool water surface elevation of 595 feet. The ultimate heat sink is designed to withstand the separate occurrence of either the safe shutdown earthquake or the PMF and is designed to withstand the effects of the postulated failure of the cooling pond dikes.

Postulated failure of the pond dikes will not erode the side slopes of the ESCP or result in loss of capacity or sediment blockage of intake facilities. Should a breach of the cooling pond dikes occur, the maximum velocity near the ESCP is estimated to be less than the nonerodible velocity of 2.0 fps. It is concluded that local levee failure will not erode the side slopes of the pond. Because of the low velocities, no scoured sediment can be transported into the ESCP, thus no loss of capacity or sediment blockage of the intake can occur as the result of local dike failure.

Over the life of the plant, it is possible that there could be some loss of storage capacity due to sedimentation in the ESCP. There are three sources which will produce sedimentation: (1) runoff from the pond drainage area, (2) erosion of the lake bottom material by the pond circulation, and (3) suspended solids contained in the makeup water from the Kankakee River. Since the runoff to the lake is negligible and the velocity of the circulating flow is small, the first two sources can be neglected as compared to the third.

Statistical analysis of 5 years of data (1957-1961) on the Kankakee River at the Wilmington gauge indicates that 50% of the time the turbidity would be lower than 15 Jackson Turbidity Units (JTU) (Reference 17). Assuming that the sediment load can be estimated from the turbidity, a sedimentation rate of 0.38 acre-feet per year could theoretically deposit in the ESCP. The total sediment deposition for a 40-year period would be 15.3 acre-feet, or 2.7% of the capacity of the ESCP. This could raise the bottom elevation to 584.17 feet. It is unlikely that all the sediment would accumulate in the ESCP, however, a periodic survey of the pond will be made to detect any change in bottom elevation.

The ESCP provides two additional station systems with water. First, the auxiliary feedwater train of each unit is tied into the essential service water piping. Second, the Seismic Category I fire protection system receives water from the ESCP. All of the above connections are provided with normally closed motor-operated valves.

2.4.12 Dispersion, Dilution, and Travel Times of Accidental Releases of Liquid Effluents in Surface Waters

Surface water bodies around the Braidwood Station buildings are the Braidwood Pond on the south, the Kankakee River on the northeast, and the Mazon River on the southwest. The town of Wilmington withdraws Kankakee River water at Wilmington, four miles and approximately three hours transit downstream of the Braidwood Station discharge. Downstream of Wilmington, the Kankakee River flows into the Des Plaines River to form the Illinois River. No other public water supplies are taken from the Kankakee, Mazon, or Illinois Rivers within 50 miles downstream from the site area.

There are only five outside surface tanks which may contain radioactive liquids. These are two identical 450,000-gallon capacity refueling water storage tanks which are located immediately west of the fuel handling building on a 6-foot thick reinforced concrete mat. The refueling water storage tank is a reinforced concrete cylindrical structure consisting of 2-foot thick walls lined on the inside with a 1/4 inch stainless steel liner. The refueling water storage tanks are considered as Category I structures (and leaktight) and are discussed in Subsection 3.8.4.

The other three tanks are the identical 500,000 - gallon capacity primary water storage tanks and the 500,000 - gallon radwaste storage tank. The primary water storage tanks are located west of the turbine building. The primary water storage tanks and tank foundations are designed for OBE and SSE seismic load conditions. The radwaste storage tank is located south and east of the Main Access Facility. A reinforced concrete wall designed for OBE seismic load conditions surrounds this non-seismic tank. The wall and surfaces contained by the wall that are susceptible to leakage are lined with a high-density polyethylene (HDPE) material. This lined enclosure is sized to contain the entire contents of the radwaste storage tank and prevent runoff to nearby surface waters or leakage into groundwater in the event of tank failure.

The largest tanks containing radioactive effluents and located outside the containment building are the 125,000-gallon recycle holdup tanks. These tanks are located in separate concrete cells with their floors at elevation 546.0 feet in the Seismic Category I auxiliary building. As described in Subsection 2.4.13.3, the maximum fluctuation in the ambient groundwater elevation is from elevation 600.0 feet to 580.0 feet. The plant grade elevation is 600.0 feet. Therefore, the only way any effluents released accidentally through postulated cracks in any component of the auxiliary building can reach a surface water body is by entering the surrounding groundwater environment.

As demonstrated in Subsection 2.4.13.3, there is no possibility that the effluents can move out of the concrete cells in the auxiliary building due to high groundwater elevation. Therefore, it is not possible for the effluents to reach a surface water body.

The Cambrian-Ordovician Aquifer is recharged in northern Illinois and southern Wisconsin. Infiltration to the Cambrian-Ordovician Aquifer in the site area is limited by the relatively impermeable Pennsylvanian-age shales of the Carbondale and Spoon Formations and by the Ordovician-age shales of the Maquoketa Shale Group. The presence of these two shale aquitards restricts the infiltration of any potential contaminants to the overlying Pleistocene age units and no potential contaminants would reach the Cambrian-Ordovician Aquifer.

2.4.13 Groundwater

Physical characteristics, yield, and water quality of the hydrogeologic units are discussed with emphasis on the major aquifers. Groundwater use and supply are also discussed.

The site work, particularly the main plant excavation and installation and testing of the construction supply water well, has confirmed that site hydrogeologic conditions were as anticipated from the PSAR-stage investigations.

2.4.13.1 Description and Onsite Use

2.4.13.1.1 Onsite Use

Groundwater is used at the Braidwood Station for potable and make-up system requirements. The location of the water supply well is shown on Figure 2.5-16, Sheet 3. The physical characteristics of the water supply well are given in Table 2.4-17. The quality of groundwater from this well is summarized in Table 2.4-18; most notable are the high concentrations of hardness, chloride, and sulfate, and total dissolved solids. A geologic log of the water supply well is presented in Table 2.4-19.

Prior to use of the water supply well installed in 2009, all plant water requirements were met by the Kankakee River following construction.

2.4.13.1.2 Site and Regional Conditions

The site area is underlain by six hydrogeologic units comprising aquifers and aquitards (confining beds). Characteristics of the units are listed in Table 2.4-20. The lithology and physical characteristics of the various stratigraphic units comprising the hydrogeologic units are described in Subsection 2.5.1.2.4.

In the vicinity of the site, Quaternary-age eolian sand, lacustrine sand, and till overlie the bedrock. The eolian and lacustrine sands are predominantly fine- to medium-grained and form a water-table sand aquifer. Many domestic water supplies in the area are obtained from the sand aquifer with well points (shallow-driven wells). The underlying glacial drift ranges from clay to sand and gravel, but is predominantly clayey till. In places, particularly in the northern part of the area, a discontinuous outwash deposit consisting mainly of silty sand and gravel serves as an aquifer within the glacial drift.

Well logs on file with the state of 80 active and nonactive wells within a few miles of the site area were studied. About one-third of the logs were for boreholes drilled for stratigraphic exploration only, for wells in areas subsequently strip-mined, or for wells which are only observation wells. There are no gas supply wells in the area. There are no wells, active or inactive, within the area of the ESCP. The logs indicate that the sand aquifer and the aquifer in the glacial drift are thin or absent in the southern part of the area and have a combined average thickness of less than 20 feet in the northern part. Analysis of boreholes on the site indicates that the thickness of the Quaternary deposits ranges from 26 to 62 feet, averaging approximately 42 feet. The saturated thickness of the sand aquifer at the site ranges from 0 to about 30 feet and averages about 14 feet. The saturated thickness of the aquifer within the glacial drift ranges from 0 to 35 feet and averages only about 5 feet thick where it is present.

Groundwater in the sand aquifer and the aquifer within the glacial drift occurs under water table conditions. These aquifers are recharged by precipitation. Groundwater is discharged from these aquifers to surface streams and strip mine pits, to the underlying bedrock, and to pumping wells. Reported well yields are suitable only for domestic or farm purposes, ranging from 2 to 5 gpm.

Chemical analyses were performed on groundwater samples collected from eight observation wells installed in the glacial drift around the main plant. Locations of the observation wells are shown on Figure 2.4-36. A summary of the results of chemical analyses performed on 15 samples collected from each observation well during 1976 are reported in Table 2.4-21. The presence of suspended solids in some samples may have interfered with the determination of representative values for some of the parameters tested in those samples.

Seven observation wells were installed in the glacial drift around the cooling pond (Figure 2.4-37). Water of notably poor quality was collected from some of these observation wells. In these samples, total dissolved solids were reported from 220 to 4332 mg/l, hardness ranged from 229 to 3160 mg/l, sulfates ranged from 50 to 2550 mg/l, and total suspended solids ranged from 4 to 443 mg/l. The differences in groundwater quality in the glacial drift between the plant site and the cooling pond may be explained by interferences resulting from sediment in the water samples and/or activities related to strip mining.

The Quaternary deposits are underlain by Pennsylvanian bedrock composed of siltstone, shale, sandstone, clay, limestone, and coal. Strip mining has removed the overlying units to the bottom of a coal horizon in the mined-out areas. The Pennsylvanian strata may locally yield up to 20 gpm from interbedded sandstones, but they are essentially aquitards, as are the underlying Maquoketa shales. The Pennsylvanian strata yield water of poor quality which is high in sulfates, chlorides, and hydrogen sulfide. Silurian dolomite, which lies below the Pennsylvanian strata and forms a shallow dolomite aquifer to the northeast and east of the site, was encountered in only two site borings.

The most important aquifer in the region is the Cambrian-Ordovician Aquifer, made up of all bedrock between the shales of the Maquoketa Shale Group and the Eau Claire Formation. The Cambrian-Ordovician Aquifer is composed of the following strata, in descending order: the Ordovician-aged Galena, Platteville, Ancell (Glenwood - St. Peter Sandstone), and Prairie du Chien Groups, and the Cambrian-aged Eminence Formation, Potosi Dolomite, Franconia Formation, Ironton Sandstone, and Galesville Sandstone. The lithology and physical characteristics of these strata are described in Subsections 2.5.1.2.4.2.3 and 2.5.1.2.4.2.4. The shales of the Maquoketa Shale Group act as a confining bed between the overlying shallow dolomite aquifer, where present, and the underlying Cambrian-Ordovician Aquifer. Groundwater in the Cambrian--Ordovician Aquifer occurs under artesian pressure. Available data indicate that, on a regional basis, the entire sequence of strata, from the top of the Galena-Platteville dolomites to the top of the Eau Claire shale beds, behaves hydraulically as one aquifer (References 18 and 19). In places, however, pressure heads between the water-bearing units differ, and the hydraulic connection is imperfect. The Cambrian-Ordovician Aquifer is recharged in northern Illinois.

An onsite groundwater well was drilled into the Cambrian-Ordovician Aquifer in 1974 to provide a construction water supply. The well was sampled to 1760 feet, was installed to 1753 feet, and is finished in the Ironton and Galesville Sandstones. The well is cased through the Maquoketa Shale Group to a depth of 280 feet (approximately elevation 320 feet MSL). The contact between the Maquoketa Shale Group and the Cambrian-Ordovician Aquifer (Galena-Platteville dolomites) was encountered at a depth of 272 feet (approximately elevation 328 feet MSL) in the well. After completion of the well, the static water level was 232 feet, indicating that groundwater in the aquifer occurs under artesian conditions. The specific capacity of the well, as determined from the pumping test, was 7.3 gpm/ft at 520 gpm after 24 hours.

The concentrations of the various parameters cannot be determined for the individual hydrogeologic units of the aquifer, since the well is open to all of the units below the Maquoketa Shale Group. However, the specific capacity and the presence of poor quality water indicate that the Ironton and Galesville Sandstones are the major producing unit. Similar conditions were encountered in the City of Braidwood Well No. 2.

The Eau Claire shales separate the Cambrian-Ordovician Aquifer from the Mt. Simon Aquifer. The Mt. Simon Aquifer includes sandstones in the lower portion of the Eau Claire Formation and the Mt. Simon Sandstone. Based on available well logs, the Mt. Simon Sandstone is anticipated at a depth of about 2400 feet below the surface. Potentiometric levels in the Mt. Simon Aquifer may be more than 50 feet higher than in the Cambrian-Ordovician Aquifer (Reference 20). Few wells in the regional area extend to the Mt. Simon Aquifer, because adequate groundwater supplies are more easily obtained from shallower aquifers, and the groundwater may be too highly mineralized for most purposes.

The construction water supply well has since been abandoned and sealed. A water supply well was subsequently installed in 2009.

2.4.13.2 Sources

2.4.13.2.1 Present and Future Groundwater Use

The water supply well is installed to a total depth of 1,750 feet below ground surface (bgs) and is cased to a depth of 1,200 feet bgs. Water entering the water supply well is derived primarily from the Ironton-Galesville aquifer. A table showing the well construction and geologic units is shown on Tables 2.4-17 and 2.4-19. The Make-up system is designed to have an average flow of approximately 100 gpm and a minimum daily net capacity of 100,000 gallons. The well is assumed to draw approximately 58 gpm daily average flow, which corresponds to daily net capacity of 83,000 gallons of water per day (gpd), up to 250 gpm average daily flow, which corresponds to daily net capacity of 360,000 gallons of water per day (gpd), in order to provide the Station with potable and industrial water. The pump itself is designed to cycle on and off at rates of over 500 gpm. An evaluation of the current use and capacity of the Cambrian-Ordovician Aquifer (specifically, the Ironton-Galesville formation or aquifer) was conducted to obtain a better understanding of the effects the Station's water supply well could have on surrounding communities. This evaluation included literature searches, water well surveys, and capture zone and estimated drawdown evaluations, as discussed below.

A literature search for data and reports discussing groundwater use in the Cambrian-Ordovician Aquifer (specifically, the Ironton-Galesville aquifer) was performed for the area surrounding the Station. These documents were reviewed in order to update the current understanding of groundwater supply and capacity in the vicinity of the Station. Maps of the potentiometric surface of the Cambrian-Ordovician Aquifer in 2000 and 2007 are presented on Figures 2.4-39 and 2.4-40, respectively. Both maps indicate a deep cone of depression in the Joliet area due to the current large-scale industrial and municipal groundwater use. However, the 2007 map indicates that this cone of depression now extends farther westward due to the installation of five new, high capacity wells in Kendall County.

As indicated on Figure 2.4-41, groundwater levels in the aquifer declined 50 to 400 feet in Kendall County, and 0 to 50 feet near Joliet and near the Braidwood Station between 2000 and 2007. The measured hydraulic gradient for the Cambrian-Ordovician Aquifer in the vicinity of the Station in 2007 was approximately 10 feet per mile (Burch, 2008). In 2000, total withdrawals from the deep bedrock aquifers within approximately 10 miles of the Station were in excess of 12 million gallons per day (interpreted from Figure 10, Wehrmann and others, 2003).

Water well surveys for wells installed in the same formations as the Station’s water supply well (deeper than 1,200 feet bgs) were conducted in order to evaluate groundwater use in the area surrounding the Station. The water well surveys identified wells installed at depths greater than 1,200 feet bgs at locations within 2, 10, and 25 miles of the Station. According to the Illinois State Geological Survey (ISGS) water well survey, there are two of these wells located within 2 miles of the Station, 71 wells within 10 miles of the Station and 201 wells within 25 miles of the Station. According to the Illinois State Water Survey (ISWS) water well survey, there are two wells located within 2 miles of the Station, 93 wells within 10 miles of the Station, and 236 wells within 25 miles of the Station. Tables 2.4-22A and 2.4-22B summarize the wells identified for the Ironton-Galesville aquifer and located within 10 miles of the Station. Tables 2.4-22C and 2.4-22D summarize these wells identified for the Ironton-Galesville aquifer and located within 25 miles of the Station. Figure 2.4-38 presents the 10- and 25-mile boundaries. Tables 2.4-23A and 2.4-23B summarize records on wells within 2 miles of the Station completed within the Ironton-Galesville aquifer.

The water supply well is open to the Ironton-Galesville aquifer and the Middle Confining Unit (see Table 2.4-19). Therefore, capture zone and drawdown calculations were conducted using the aquifer properties of the Ironton-Galesville aquifer only. Table 2.4-20 presents several values for hydraulic conductivity, hydraulic gradient, and thickness of the Ironton-Galesville aquifer obtained from various studies conducted in the northern Illinois area. The maximum width of the capture zone for the Station's water supply well is approximately 1,210 feet in each cross-gradient direction at over 8,200 feet upgradient. The drawdown at this distance from the well was determined to be negligible since it is well outside the 0.5 feet drawdown radius, as discussed below. The capture zone was calculated to extend approximately 390 feet downgradient from the well. Figure 2.4-42A indicates the orientation and dimensions of the capture zone. This figure also depicts the relationship between the predicted capture zone, nearby water supply wells, and other site features.

The Theis Equation (Fetter, 1994) was used to predict drawdown surrounding the Station’s water supply well. These calculations assume steady state conditions and a constant average pumping rate from the Station's water supply well. Figure 2.4-42B presents the predicted drawdown at various distances from the water supply well. Based on the aquifer properties presented in Table 2.4-20 and an aquifer storativity of 0.01, the drawdown is generally less than 1 foot beyond 500 feet away from the pumping well and less than half a foot at a distance of 1,000 feet away from the pumping well. In summary, the "radius of influence" of the water supply well is not expected to be much more than 1,000 feet at a constant pumping rate of 58 gpm.

Simple analytical techniques were used to evaluate the pumping influence of the well on the deep Ironton-Galesville aquifer and on identified wells completed within the same aquifer system. These analytical techniques assumed reasonable worst case scenarios of pumping of the Station's water supply well by assuming constant or steady state conditions. The well inventory was comprehensive and yet, only identified two wells in the same aquifer within 2 miles of the Station. The hydraulic analyses identified that the amount of drawdown that is measureable and therefore significant from the pumping of the Station’s water supply well at the 58 gpm rate would not extend more than 1,000 feet from the well. This distance (the "radius of influence") is still within the confines of the site area boundary. The analyses also identified the zone of groundwater within the Ironton-Galesville aquifer to be captured from the pumping of the water supply well. This "capture zone" is developed from the pumping of the Station’s water supply well on the regional gradient, or natural flow, within the same aquifer. The results of the capture zone analysis indicate that groundwater located at a distance of greater than 390 feet downgradient (to the northeast) is not impacted by pumping of the Station's water supply well. The well inventory identified, in April 2010, only two water supply wells (completed within the same Ironton-Galesville aquifer) within a 2-mile radius of the Station’s well.

These two water supply wells are also located to the north and downgradient from the Station’s water supply well. One of these off property wells is located over 6,000 feet from the water supply well. The other well is located over 10,000 feet from the water supply well and is located on the edge of the 2-mile radius. These wells are used by the Village of Braidwood. Based upon the above information on current well use and on the pumping influence of the Station’s water supply well, it is concluded that the groundwater pumping will have a negligible to small impact on groundwater users. The nearest well completed in the deep aquifer is over 6,000 feet away from the Station’s water supply well. It was determined that the radius of influence and the capture zone of this downgradient, Village of Braidwood, well would not extend upgradient to the radius of influence and capture zone of the Station’s water supply well. As such, the evaluations indicate that there will be no reduction of yield of the two Village of Braidwood wells identified in April 2010.

This supports the conclusion that Station’s water supply well pumpage will have a negligible to small impact on nearby groundwater users identified in April 2010. An additional evaluation of the potential of the Station's water supply well or other wells pumping in the same area to impact groundwater users has also been performed. This hypothetical evaluation assumed that a higher pumping rate was established by a well or wells located at the same location of the water supply well. This evaluation was completed using the same aquifer and hydraulic parameters provided in the previous sections and the same analytical methods. It was determined that the Station's water supply well or additional wells would need to pump at an average (annual, constant or steady state pumping) rate of over 250 gpm in order to establish a downgradient capture zone that extended beyond the site area northern boundary (approximately 1,600 feet). The drawdown calculations indicated that this average pumping rate would result in a steady state drawdown of less than one foot at a distance of 1,600 feet, near the site area boundary. In summary, the additional hydraulic evaluations indicate that pumping (of a well or wells at the water supply well location) at a rate up to approximately 250 gpm (or 360,000 gpd), under current hydrogeologic conditions, would not significantly capture or measurably influence groundwater located off of the site area boundary. As such, pumping of a well or wells at this rate would also be considered to have a negligible to small impact on nearby groundwater users identified in April 2010.

2.4.13.2.2 Site Hydrogeologic Conditions

2.4.13.2.2.1 Permeability

Permeability values for the various hydrogeologic units at the site were determined from laboratory tests on soil samples, field permeability tests conducted in the ESCP area, and water pressure tests in the bedrock. These tests are described in Subsections 2.5.4.6 and 2.5.6.

Laboratory permeability test results, reported in Table 2.5-24, show the permeability of the sand deposits to range from 3.66 X 10^{-4} cm/sec to 7.32 X 10^{-2} cm/sec. For the evaluation of seepage from the ESCP, an average value of 6 X 10^{-3} cm/sec was used. The average permeability of the till was found to be 2.6 X 10^{-6} cm/sec. For discontinuous, well-graded gravel and silts within the glacial drift at a depth of 35.5 to 40.5 feet in Borings H-1 and H-3, the permeability was found to average 8.4 X 10^{-4} cm/sec.

The results of the water pressure tests conducted in bedrock are presented in Tables 2.5-20 through 2.5-23 and on boring logs presented in Figure 2.5-159. Water pressure tests were performed in the Pennsylvanian-age Carbondale and Spoon Formations and in the underlying Brainard Shale and Fort Atkinson Limestone of the Ordovician-age Maquoketa Shale Group. No water losses were recorded in 20% and 50% of the tested intervals in the Carbondale and Spoon Formations, respectively, or in 40% of the tested intervals in the Maquoketa Shale Group. In those intervals in which water losses were recorded, permeabilities ranged from 1.93 X 10^{-6} to 4.92 X 10^{-4} cm/sec in the Carbondale Formation, 1.76 X 10^{-6} to 6.20 X 10^{-4} cm/sec in the Spoon Formation, and 2.33 X 10^{-6} to 4.58 X 10^{-5} cm/sec in the Maquoketa Shale Group. These permeability values probably reflect secondary permeability along infrequent joints and fractures within these formations rather than intergranular, primary permeability of the rock mass. In addition, the upper tested intervals of the boreholes generally had higher permeabilities than those at greater depths, probably reflecting the effects of weathering on the strata.

2.4.13.2.2.2 Groundwater Levels

Groundwater levels at the time the borings were drilled in the plant area (January 1973 to April 1973) were at approximately elevation 595 feet. Groundwater levels recorded in piezometers installed during the site investigations are presented in Table 2.5-35. Locations of these piezometers are shown on Figure 2.5-16.

Eight observation wells were installed in the glacial drift around the power block excavation and outside the slurry trench in late 1975 to monitor groundwater levels during construction. These observation wells were installed in pairs at varying distances away from the slurry trench. The locations of these observation wells are shown on Figure 2.4-36, and a typical detail of an observation well is shown on Figure 2.4-43. Groundwater levels measured in these observation wells are shown on Figure 2.4-44.

Seepage from the sand aquifer into the power block excavation was limited by a slurry trench installed from approximately elevation 595 feet MSL to 2 feet into the till underlying the sand aquifer. The combined quantities of seepage and precipitation were controlled using a sump pump. For approximately 77% of the measurements, groundwater levels were higher in the outer observation well of each pair, indicating some decline of groundwater levels immediately adjacent to the slurry trench due to seepage into the excavation. The average difference in groundwater levels between pairs of observation wells was 0.7 foot. The slight decline in groundwater levels and the small volume of seepage into the excavation indicate that groundwater levels in the sand aquifer were affected only in the immediate proximity of the power block excavation.

Groundwater levels have also been measured since July 1973 in seven shallow observation wells installed around the cooling pond (Figure 2.4-45). Average monthly groundwater levels have been plotted from July 1973 through December 1978 for each observation well on Figure 2.4-45. In general, the highest groundwater levels were observed from January through June, and the lowest groundwater levels from July through December. These seasonal variations are not as evident in observation well LW-1 because the groundwater levels were influenced by water levels in a nearby ditch which carries water pumped from a gravel pit located offsite. Infiltration from the ditch resulted in temporarily high groundwater levels during the summer months of 1974, 1975, and 1976. Over the period of record, groundwater levels in the pond observation wells fluctuated within a range of 0 to 9 feet below the ground surface, except in observation well LW-1, where the groundwater level ranged from 7.5 feet to 14.8 feet below the ground surface. The difference in groundwater elevations between the observation wells can be attributed to the varying ground surface elevations, local drainage patterns, and other factors relating to man's activities, such as pumpage from nearby mine pits.

2.4.13.2.3 Effects of Seepage from Cooling Pond

Seepage from the cooling pond should have minimal effect on groundwater levels around the site. Seepage to the Cambrian-Ordovician Aquifer is limited by the relatively impermeable Pennsylvanian-age shales of the Carbondale and Spoon Formations and by the Ordovician-age shales of the Maquoketa Shale Group. Seepage to the sand aquifer will be limited by a slurry trench cutoff, constructed through the cooling pond dike and generally extending 2 feet into the till or onto Pennsylvanian-age bedrock. The cooling lake perimeter dike slurry trench is continuous around the perimeter of the cooling lake and, therefore, continuous along the north and west sides of the essential service cooling pond (ESCP). Plan views of the ESCP and perimeter dike along the north and west sides of the ESCP are given in Figures 2.4-26 through 2.4-29. Sections of the perimeter dike and slurry trench are given in Figure 2.4-35. The slurry trench along the ESCP is a soil-bentonite backfilled slurry trench extending from elevation 597 feet to the top of the till and, in most cases, is keyed into the till. As-built profiles are provided in Figures 2.4-49, 2.4-50 and 2.4-51. The slurry trench will be a continuous seepage cutoff around the entire perimeter of the pond. However, the design of the ESCP does not rely on the slurry trench as a seepage barrier. The ESCP seepage has been conservatively determined (see Subsection 2.5.6) assuming the trench does not exist.

As part of the design evaluation for the slurry trench cutoff, a prototype slurry trench test section was constructed in the cooling pond area. The test consisted of several pumping tests to determine the average permeability of the in-place soil-bentonite and cement-bentonite backfill materials. The results of these tests were used in the design of the slurry trench cutoff and the cooling pond dike (Subsection 2.5.6). The maximum permeability values determined for the in-place slurry trench test section are as follows:

Soil-bentonite (using natural onsite soil)	6.0×10^{-7} cm/sec
Cement-bentonite	4.4×10^{-6} cm/sec

Based upon these permeability values, the amount of seepage through the entire length of the cooling pond dike is estimated to be less than 5 cfs (Subsection 2.5.6). Therefore, when considering the approximately 10-mile perimeter of the cooling pond, the effect on local groundwater levels in the sand aquifer should be very small and restricted to the immediate perimeter of the cooling pond.

Temperature and total dissolved solids concentration of the pond water may exceed those of the groundwater in the sand aquifer. However, any effects of seepage of water of different temperature or chemistry into the sand aquifer should be minimal due to dilution of the small quantity of seepage in the rainfall-recharged aquifer. Dissolved solids concentration in the pond will be held below effluent discharge standards by blowdown to the Kankakee River.

2.4.13.2.4 Seepage from the Essential Service Cooling Pond

The results of a seepage analysis performed to determine the seepage losses from the ESCP in the unlikely event of a failure of the main perimeter cooling pond dike and loss of all pond water above elevation 590 feet MSL are presented in Subsection 2.5.6.

2.4.13.3 Accident Effects

As described in Subsection 2.4.12, the largest tanks which are located outside the containment building and contain radioactive effluents are the boron recycle holdup tanks. Each of these tanks has a capacity of 125,000 gallons. The floor and roof elevations of the concrete cells in which these tanks are located are 546.0 and 583.0 feet, respectively. The plant grade elevation is 600.0 feet.

The porous media surrounding the building are comprised of Parkland Sand and Equality Formation from elevation 600.0 feet to 580.0 feet, underlain by about 20 feet of Wedron Formation. The average permeability of the Parkland Sand ranges from 1.2×10^{-5} to 2.4×10^{-3} ft/sec. The average permeability of the Wedron Formation is 8.53×10^{-8} ft/sec (see Subsections 2.4.13.1.2 and 2.4.13.2.2).

A cement bentonite slurry trench has been installed around the perimeter of the main plant excavation through the Parkland Sand and the Equality Formation into the silty clay glacial till of the Wedron Formation. This trench would restrict any seepage into or out of the auxiliary building.

The nearest groundwater user is located about 1850 feet from the auxiliary building (Well no. 73 on Figure 2.4-42).

The design groundwater elevation at the plant site is 600.0 feet. History of groundwater withdrawal from the Parkland Sand indicates that this aquifer has been continually supplying water with no evidence of any serious depletion. Therefore, it is reasonable to assume that the water table fluctuates within the Parkland Sand and Equality Formation and will always be above elevation 580.0 feet.

To examine the impact of a postulated accidental release of radioactive effluents, it is hypothesized that one of the boron recycle holdup tanks spills its contents into the concrete cell in which it is located. The walls or foundation of this cell are postulated to develop some cracks through which direct communication is established between the interior of the building and the surrounding groundwater environment. The maximum elevation of the spilled fluid inside the cell is estimated to be 563.0 feet. As the ambient groundwater elevation is 17 to 37 feet higher than the fluid level inside the cell, there would be no hydraulic gradient from the interior of the cell to the outside. Therefore, the effluents will be contained and prevented from moving out of the building and contaminating the surrounding groundwater environment.

2.4.13.4 Monitoring

Groundwater monitoring has been performed in eight observation wells installed in the sand aquifer around the perimeter of the main plant excavation (Figure 2.4-36). Monitoring has included groundwater levels and quality. The results of this monitoring have shown no significant effects on groundwater levels or quality due to minor seepage into the plant excavation (Braidwood OLS-ER, Sections 2.4.2.1.2 and 2.4.2.2.1 and Braidwood UFSAR, Figure 2.4-44). The excavation has been backfilled to essentially the final plant grade. Monitoring will continue on a quarterly basis at station wells PW-2 and PW-6, and will be discontinued one year after the pond is filled.

Groundwater monitoring has been performed in seven observation wells installed in the glacial drift around the cooling pond (Figure 2.4-37). Groundwater levels are plotted on a time scale in Figure 2.4-45. Monitoring will continue on a quarterly basis on those wells that are accessible and will be discontinued one year after the pond is filled. 71 additional observation wells were installed during the 1978 construction season to monitor any effects of pond filling on groundwater levels. Additional observation wells may be added based on evaluation of the results of the 1979 construction season.

The frequency of monitoring groundwater levels in the 71 observation wells will be twice monthly prior to pond filling. Starting with pond filling, groundwater levels will be monitored weekly until one month after pond filling. Groundwater levels will then be monitored twice monthly to one year after pond filling. Thereafter, groundwater levels will be measured monthly.

Observation well data collected since December 1980 has indicated the cooling pond and dike system is functioning as anticipated with no indications of significant seepage problems. Based on the evaluation of these results, the number of observation wells to be monitored on a monthly basis during plant operation will be reduced to 40.

2.4.13.5 Design Bases for Subsurface Hydrostatic Loading

The design groundwater level for hydrostatic loading is elevation 600 feet MSL. Foundations of the main plant buildings are below the groundwater levels measured in the sand aquifer during site investigations and construction (Subsections 2.4.13.2.2.2 and 2.5.4.6). Seepage into the main plant excavation was controlled during construction by a slurry wall cutoff installed through the sand aquifer and 2 feet into the underlying till. Precipitation and seepage into the excavation were removed by the use of sump pumps. There are no dewatering systems that will be used to permanently lower groundwater levels under safety- or non-safety-related buildings.

2.4.14 Technical Specification and Emergency Operation Requirements

2.4.14.1 Probable Maximum Flood Level

The probable maximum flood level for Braidwood Station, as determined in Subsection 2.4.3 and summarized in Table 2.4-5, is 598.17 feet MSL due to the flood stages at the cooling pond. The probable maximum flood stage of the Kankakee River is 561.3 feet. Since the plant floor elevation is 601.0 feet, these floods will have no damaging effect on any safety-related structure. The probable maximum precipitation at the plant site, as discussed in Subsection 2.4.2.3, will cause only minor local flooding and will not have appreciable effect on the perched groundwater elevation. This local flood will reach an elevation higher than the plant floor elevation for a short period, and as discussed in Subsection 2.4.2.3, special provisions are made for safety-related facilities to prevent flooding. See Table 2.4-24 for safety-related structures and pertinent design water levels.

2.4.14.2 Flood Protection Measures

As mentioned in Subsection 3.4.1, the probable maximum flood stage (elevation 598.17) is below the plant grade elevation (elevation 600). As an additional safety measure, the following safeguards are provided:

a. Waterstops are provided in all horizontal and vertical construction joints in all exterior walls up to the grade elevation.

b. Water seals are provided for all penetrations in exterior walls up to the grade elevation.

c. All exterior walls and the base mat are designed for the hydrostatic pressure considering submergence to the probable maximum flood elevation.

d. The finished grade elevation adjacent to the plant is maintained at least 1.0 foot below plant grade floor elevation.

e. No exterior access openings are provided below the grade floor elevation, in areas where flooding could damage Safety Class 1 structures or equipment.

2.4.14.3 Emergency Protective Measures

There are no requirements for emergency protective measures designed to minimize the impact of hydrology-related events on safety-related facilities, and none were incorporated into the technical specification and emergency procedures. The ultimate heat sink is a man-made pond which is not subject to degradation due to a hydrology-related event.

2.4.15 References

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TABLE 2.4-1

FLOODS ON THE KANKAKEE RIVER NEAR WILMINGTON

WATER YEAR	PEAK FLOOD DISCHARGE (cfs)	PEAK FLOOD STAGE (ft)	MAXIMUM GAUGE HEIGHT (ft)
1981	41,000	6.45	Same
1980	24,800	5.88	Same
1979	48,000	--	12.07
1978	30,500	6.68	9.40
1977	16,200	4.54	Same
1976	32,600	6.95	Same
1975	27,100	6.24	Same
1974	49,100	8.49	12.78
1973	33,200	7.03	Same
1972	15,800	4.47	Same
1971	12,600	4.07	Same
1970	54,500	9.40	Same
1969	29,700	6.59	Same
1968	35,100	7.26	13.88
1967	19,400	5.18	10.08
1966	23,400	5.75	6.99
1965	19,500	5.20	Same
1964	10,800	3.70	Same
1963	22,000	--	9.72
1962	23,800	5.70	6.68
1961	17,000	4.86	Same
1960	19,500	5.25	9.13
1959	30,000	--	9.52
1958	30,600	6.72	9.92
1957	75,900	11.40	Same
1956	16,200	4.70	Same
1955	14,400	4.38	7.13
1954	15,000	4.53	Same
1953	19,500	5.17	Same
1952	29,000	6.46	9.43
1951	30,000	--	10.83
1950	37,800	7.61	11.39
1949	16,700	4.8	11.57
1948	23,000	5.67	6.00
1947	21,600	5.40	Same
1946	19,500	5.2	--
1945	21,600	5.4	--
1944	33,800	7.1	--
1943	48,000	8.87	10.06
1942	46,600	8.7	--
1941	8,290	3.30	--

TABLE 2.4-1 (Cont'd)

	PEAK FLOOD		MAXIMUM GAUGE
WATER YEAR	DISCHARGE (cfs)	STAGE (ft)	HEIGHT (ft)
1940	11,100	3.95	--
1939	24,600	6.0	--
1938	19,600	5.3	--
1937	15,100	4.65	--
1936	17,500	5.0	--
1935	17,500	5.0	--
1934	7,000	--	--
1933	35,300	--	--
1932	10,600	--	--
1931	6,510	--	--
1930	17,200	--	--
1929	24,800	--	--
1928	24,000	--	--
1927	29,100	--	--
1926	20,900	--	--
1925	14,100	--	--
1924	18,900	--	--
1923	16,400	--	--
1922	34,300	--	--
1921	7,270	--	--
1920	26,200	--	--
1919	22,800	--	--
1918	26,600	--	--
1917	15,600	--	--
1916	14,500	--	--
1915	22,400	--	--
1887	--	--	16.73
1883	--	--	16.73

TABLE 2.4-2

PROBABLE MAXIMUM PRECIPITATION ON THE POND BASIN

STORM DURATION (hr)	PMP (in.)
6	24.4
12	26.7
24	29.6
48	31.9

TABLE 2.4-3

PROBABLE MAXIMUM PRECIPITATION DISTRIBUTION

STORM HOUR	INCREMENTAL PRECIPITATION (in.)	CUMULATIVE PRECIPITATION (in.)
1	1.0	1.0
2	1.9	2.9
3	3.2	6.1
4	11.6	17.7
5	5.1	22.8
6	1.6	24.4
7	0.5	24.9
8	0.5	25.4
9	0.4	25.8
10	0.4	26.2
11	0.3	26.5
12	0.3	26.8
13	0.3	27.1
14	0.3	27.4
15	0.3	27.7
16	0.3	28.0
17	0.2	28.2
18	0.2	28.4
19	0.2	28.6
20	0.2	28.8
21	0.2	29.0
22	0.2	29.2
23	0.2	29.4
24	0.2	29.6
25	0.15	29.75
26	0.15	29.90
27	0.15	30.05
28	0.15	30.20
29	0.10	30.30
30	0.10	30.40
31	0.10	30.50
32	0.10	30.60
33	0.10	30.70
34	0.10	30.80
35	0.10	30.90
36	0.10	31.00
37	0.10	31.10
38	0.10	31.20
39	0.10	31.30
40	0.10	31.40
41	0.10	31.50
42	0.10	31.60
43	0.05	31.65

TABLE 2.4-3 (Cont'd)

STORM HOUR	INCREMENTAL PRECIPITATION (in.)	CUMULATIVE PRECIPITATION (in.)
44	0.05	31.70
45	0.05	31.75
46	0.05	31.80
47	0.05	31.85
48	0.05	31.90

TABLE 2.4-4

MAXIMUM RAINFALL INTENSITY DURING LOCAL PROBABLE MAXIMUM PRECIPITATION

TIME (minutes)	CUMULATIVE RAINFALL (inches)	RAINFALL INTENSITY (inches per hour)
5	5.98	71.8
15	9.47	37.9
30	13.56	27.1
60	17.8	17.8

TABLE 2.4-5

PROBABLE MAXIMUM FLOOD AND OTHER ELEVATIONS

LOCATION	MAXIMUM ELEVATION (ft)
Pond PMF	598.17
Kankakee River at intake:	
- PMF	561.30
- low flow	534
- average annual flow	538
- flood of record	552
Mazon River at old Highway 66	582
Granary Creek just upstream of East Fork Mazon River	576

TABLE 2.4-6

PROBABLE MAXIMUM PRECIPITATION ON CRANE AND GRANARY CREEKS, MAZON AND KANKAKEE RIVERS

HOUR	CRANE AND GRANARY CREEKS, 52.2 mi^2 (in.)	MAZON RIVER, 220 mi^2 (in.)	KANKAKEE RIVER, 5150 mi^2 (in.)
3	0.15	0.25	
6	0.2	0.25	12.50
9	0.20	0.3	
12	0.25	0.3	2.10
15	0.30	0.4	
18	0.35	0.4	0.86
21	0.45	0.5	
24	0.50	0.7	0.54
27	0.90	1.9	
30	1.90	12.8	0.54
33	15.50	4.2	
36	5.20	1.1	0.60
39	1.20	0.6	
42	0.60	0.4	0.48
45	0.50	0.4	
48	0.40	0.3	0.48

TABLE 2.4-7

BASIN CHARACTERISTICS FOR CRANE AND GRANARY CREEKS AND MAZON RIVER

ITEM	CRANE AND GRANARY CREEK	MAZON RIVER
Drainage area (mi^2)	52.2	220.0
Main stream length (mi)	18.1	22.0
Approximate channel elevation at down-steam end (ft)	562.0	556.0
C_t	5.0	3.7
$640C_p$	320.0	530.0
Unit duration for unitgraph (hr)	3.0	3.0
Time to peak for unitgraph (hr)	20.2	19.4
Peak discharge for unitgraph (cfs)	830.0	6000.0

TABLE 2.4-8

FLOOD ELEVATIONS

CROSS SECTION	DRAINAGE AREA (mi^2)	PMF (cfs)	WATER SURFACE ELEVATION (ft)
X-GD	23.1	8,630	586.5
X-GU	21.6	8,070	594.5
X-CD	26.4	9,860	587.0
X-CU	25.3	9,450	591.5
X-MU1	112.0	57,000	587.0
X-MU2	117.0	59,500	586.0
X-MD1	215.0	109,500	582.5
X-MD2	220.0	112,000	581.5

TABLE 2.4-9

PROBABLE MAXIMUM FLOOD CHARACTERISTICS FOR THE POND

Drainage area (mi^2)	5.3
Normal pool elevation (ft)	595.0
Pond area at normal pool (mi^2)	3.9
PMP duration (hr)	48.0
PMF volume (acre-ft)	9,050.0
PMF peak inflow (cfs)	39,600.0

TABLE 2.4-10

SPILLWAY RATING TABLE

WATER SURFACE ELEVATION (ft)	HEAD (ft)	DISCHARGE (cfs)
596.00	0.25	73
596.25	0.50	205
596.50	0.75	377
596.75	1.00	610
597.00	1.25	811
597.25	1.50	1075
597.50	1.75	1342
597.75	2.00	1648
598.00	2.25	1954
598.25	2.50	2290

Note: Bases:

1. 200 foot wide broad-crested spillway.
2. Trapezoidal shape.
3. 10:1 inclined floor at downstream.
4. Spillway crest elevation 595.75 feet.

TABLE 2.4-11

WIND-WAVE CHARACTERISTICS ON THE BRAIDWOOD POND - DESIGN-BASIS WIND

						WIND TIDE SETUP			WAVE RUNUP					TOTAL SETUP PLUS RUNUP (ft)
									WAVE HEIGHT (ft)		RUNUP (ft)			
											**	***		
LOCATION*	FLOOD AND WIND CONDITION	WIND SPEED OVER LAND (mph)	WIND SPEED OVER WATER (mph)	AVERAGE WATER DEPTH (ft)	EMBANKMENT SLOPE (%)	FETCH (mi)	SETUP (ft)	EFFECTIVE FETCH (mi)	SIG. H_s	MAX. H_{max}	w/H_s	w/H_{max}	w/H_s	w/H_{max}
A	SPF+PMF +40-mph Wind	40	46.0	11.46	33.3	2.50	0.33	1.25	2.35	3.92	2.77	3.84	3.10	4.17
B	PMF +25-mph Wind	25	28.5	10.15	33.3	2.14	0.12	1.07	1.33	2.22	1.65	--	1.77	--
C	PMF +25-mph Wind	25	28.7	11.15	33.3	1.95	0.10	1.22	1.35	2.25	1.70	--	1.80	--
D	PMF +25-mph Wind	25	28.0	11.35	33.3	1.18	0.06	0.93	1.15	1.92	1.31	--	1.37	--
E	PMF +25-mph wind	25	27.5	7.91	33.3	1.34	0.09	0.67	1.03	1.72	1.32	--	1.41	--

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* For locations refer to Figure 2.4-34.

** w/H_s - with significant wave height.

*** w/H_{max} - with maximum wave height.

TABLE 2.4-12

DIKE FREEBOARD - DESIGN-BASIS WIND

LOCATION	FLOOD AND WIND CONDITION	TYPE OF WAVE	NORMAL POOL ELEVATION (ft MSL)	PMF MAXIMUM RISE (ft)	SETUP PLUS RUNUP (ft)		WATER SURFACE RUNUP ELEVATION (ft MSL)		TOP ELEVATION OF DIKE
					w/H_s	w/H_{max}	w/H_s	w/H_{max}	
A	SPF + PMF + 40-mph Wind	Shallow	595.0	3.17	3.10	4.17	601.27	602.34	602.50
B	PMF + 25-mph Wind	Shallow	595.0	2.91	1.77	---	599.68	---	600.00
C	PMF +25-mph Wind	Shallow	595.0	2.91	1.80	---	599.71	---	600.00
D	PMF + 25-mph Wind	Deep	595.0	2.91	1.37	---	599.28	---	600.00
E	PMF +25-mph Wind	Shallow	595.0	2.91	1.41	---	599.32	---	600.00

TABLE 2.4-13

WIND-WAVE CHARACTERISTICS ON THE BRAIDWOOD POND - EXTREME WIND

						WIND TIDE SETUP		WAVE RUNUP					TOTAL SETUP PLUS RUNUP (ft)	
									WAVE HEIGHT (ft)		RUNUP (ft)			
											**	***		
LOCATION*	FLOOD AND WIND CONDITION	WIND SPEED OVER LAND (mph)	WIND SPEED OVER WATER (mph)	AVERAGE WATER DEPTH (ft)	EMBANKMENT SLOPE (%)	FETCH (mi)	SETUP (ft)	EFFECTIVE FETCH (mi)	SIG. H_s	MAX. H_{max}	w/H_s	w/H_{max}	w/H_s	w/H_{max}
A	Extreme wind + normal pool	60	64.8	11.00	33.3	0.61	0.17	0.45	2.30	3.84	2.74	3.76	2.91	3.93
B	Extreme wind + normal pool	60	68.4	7.24	33.3	2.14	0.99	1.07	2.66	4.44	2.93	--	3.92	--
C	Extreme wind + normal pool	60	69.0	8.24	33.3	1.95	0.80	1.22	2.95	4.93	3.25	--	4.05	--
D	Extreme wind + normal pool	60	67.2	8.44	33.3	1.18	0.45	0.93	2.76	4.61	3.12	--	3.57	--
E	Extreme wind + normal pool	60	66.0	5.00	33.3	1.34	0.83	0.67	2.11	3.52	2.32	--	3.15	--

---

* For locations refer to Figure 2.4-34.

** w/H_s - with significant wave height.

***w/H_{max} - with maximum wave height.

TABLE 2.4-14

DIKE FREEBOARD - EXTREME WIND

LOCATION	FLOOD AND WIND CONDITION	TYPE OF WAVE	NORMAL POOL ELEVATION (ft MSL)	PMF MAXIMUM RISE (ft)	SETUP PLUS RUNUP (ft)		WATER SURFACE RUNUP ELEVATION (ft MSL)		TOP ELEVATION OF DIKE
					w/H_s	w/H_{max}	w/H_s	w/H_{max}	
A	Extreme wind + normal pool	Shallow	595.0	---	2.91	---	597.91	---	602.50
B	Extreme wind + normal pool	Shallow	595.0	---	3.92	---	598.92	---	600.00
C	Extreme wind + normal pool	Shallow	595.0	---	4.05	---	599.05	---	600.00
D	Extreme wind + normal pool	Shallow	595.0	---	3.57	---	598.57	---	600.00
E	Extreme wind + normal pool3	shallow	595.0	---	3.15	---	598.15	---	600.00

TABLE 2.4-15

STAGE/FLOW DATA AT CUSTER PARK AND WILMINGTON

YEAR	AVERAGE STAGE AT CUSTER PARK (ft)	AVERAGE FLOW AT WILMINGTON (cfs)
1971	537	3200
1970	538	4800
1965	538	4400
1964	537	1500

TABLE 2.4-16

LOW FLOW FREQUENCY/DURATION DATA

KANKAKEE RIVER

FREQUENCY/DURATION	MEAN FLOW AT WILMINGTON GAUGE (cfs)	MEAN FLOW AT RIVER SCREEN HOUSE (cfs)
1-day 10-year low	389	378
3-day 10-year low	428	415
7-day 10-year low	453	440
30-day 10-year low	509	494
1-day 100-year low	270	262
3-day 100-year low	305	296
7-day 100-year low	331	321
30-day 100-year low	396	385
Historical low flow	204	198
Average flow	4,071	3,952

TABLE 2.4-17

PHYSICAL CHARACTERISTICS OF THE WATER SUPPLY WELL

Location, plant coordinates	41 deg. 14 min. 43 sec.NORTH 88 deg. 13 min. 52 sec.WEST
Date completed	4/22/2009
Surface elevation	Approx. 600 ft MSL
Total depth	1750 ft
Deepest hydrogeologic unit encountered	Ironton and Galesville Sandstones
Depth to bottom of casing	1200 ft
Lowest formations cased	Knox Megagroup
Pumping test data	
Date test began	3/29/2010
Static water level	288 ft
Pumping water level	335 ft
Pumping rate	610 gpm
Length of pumping test	24 hr
Specific capacity	13.0 gpm/ft

TABLE 2.4-18

PARTIAL WATER QUALITY ANALYSES FOR WATER SUPPLY WELL*

PARAMETER*	RESULT	
pH (at 25° C)	7.46	SU
Hardness (as CaCO3)	511	mg/L
Alkalinity (as CaCO3)	242	mg/L
Chloride	308	mg/L
Sulfate	488	mg/L
Sodium	308	mg/L
Fluoride	1.15	mg/L
Iron	0.18	mg/L
Nitrate	<0.07	mg/L
Nitrite	<0.15	mg/L
Total dissolved solids	1471	mg/L
Silica	9.01	mg/L
Potassium	27.9	mg/L
Calcium	121	mg/L
Magnesium	50.8	mg/L
Aluminum	<37	ug/L
Arsenic	15.5	ug/L
Barium	15.2	ug/L
Beryllium	0.55	ug/L
Boron	1120	ug/L
Chromium	<5.8	ug/L
Cooper	1.95	ug/L
Manganese	5.6	ug/L
Nickel	38	ug/L
Zinc	43.3	ug/L
Gross Alpha	<57.6	pCi/L
Tritium	<203	pCi/L

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[*] Water samples were collected between August 2009 and March 2010, during test pumping of the well.

TABLE 2.4-19

GEOLOGIC LOG, WATER SUPPLY WELL

SYSTEM	SERIES	GROUP OR FORMATION	DEPTH (ft)	THICKNESS (ft)	HYDROGEOLOGIC UNIT
Quaternary	Pleistocene	Undifferentiated	48	48	Glacial drift Aquifer
Pennsylvanian	Desmoinesian	Undifferentiated	105	57	Pennsylvanian Aquitard
Ordovician	Cincinnatian	Maquoketa Shale Group	270	165	Maquoketa Aquitard
		Galena Group	580	310	Cambrian-Ordovicion Aquifer
		Platteville Group	655	75	
		Ancell Group (Glenwood-St. Peter Sandstone)	1137	482	
	Canadian	Knox Mega Group	1450	313	

TABLE 2.4-19 (Cont'd)

GEOLOGIC LOG, WATER SUPPLY WELL (Continued)

SYSTEM	SERIES	GROUP OR FORMATION	DEPTH (ft)	THICKNESS (ft)	HYDROGEOLOGIC UNIT
Cambrian	Croixan	Franconia Formation	1580	130	
		Ironton and Galesville Sandstones	1745	165	
		Eau Claire Formation	1750	5+	

NOTES

1. Well drilled by Richard W. Wehling, completed on April 22, 2009.
2. Well located Sec 19, T.32N., R. 9E. in Will County.
3. Drillers log indicated the top of bedrock was encountered at 48 feet.
4. Geologic log is based upon the driller’s log.
5. Location of construction supply well is shown on Figure 2.5-16.

TABLE 2.4-20

STRATIGRAPHIC UNITS AND THEIR HYDROGEOLOGIC CHARACTERISTICS

SYSTEM	SERIES	GROUP OR FORMATION	HYDROGEOLOGIC UNIT		DESCRIPTION	HYDROGEOLOGIC CHARACTERISTICS
QUATERNARY	Pleistocene	Parkland Sand	Eolian sand	Sand Aquifer	Silty fine sand	Groundwater occurs in the sand formations under water table conditions, perched on the underlying till. Groundwater also occurs in the outwash layers within the till. The small thickness of the upper sand and the discontinuous nature of the outwash preclude extensive development of the sand aquifer or the aquifer within the till.
		Equality Formation	Lacustrine sand	Sand Aquifer	Fine to medium sand with trace to little silt	
		Wedron Formation	Till	Aquitard	Silty clay, clayey silt, and sandy silt with interspersed sand and gravel, some discontinuous layers of gravelly sand or sandy gravel	
PENNSYLVANIAN	Desmoinesian	Carbondale Formation	Pennsylvanian siltstone	Aquitard	Principally siltstone, with some interbedded shale, underclay, sandstone, limestone, and coal	Groundwater occurs primarily in thin sandstone beds and occasionally in joints in thin limestone beds. Ground water occurs under leaky artesian conditions. The high proportion of siltstone makes the Pennsylvanian strata generally unfavorable as an aquifer.
		Spoon Formation	Pennsylvanian siltstone	Aquitard		

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*The table is modified from Illinois EPA (1976) and Sasman et al. (1976). A detailed discussion of the lithology and physical characteristics of the various stratigraphic units is presented in Subsection 2.5.1.8.4.

TABLE 2.4-20 (Cont.)

SYSTEM	SERIES	GROUP OR FORMATION	HYDROGEOLOGIC UNIT		DESCRIPTION	HYDROGEOLOGIC CHARACTERISTICS
						Yields are low and are suitable only for domestic and farm purposes.
SILURIAN	Alexandrian	Undifferentiated	Silurian dolomites	Shallow Dolomite Aquifer	Dolomite with thin shale partings, and dolomitic siltstone	Groundwater occurs primarily in joints in the dolomites and limestones under leaky artesian conditions. The shales are generally not water yielding and act as confining beds between the shallow and deep aquifers.
ORDOVICIAN	Cincinnatian	Maquoketa Shale Group	Maquoketa shale	Aquitard	Silty dolomitic shale at top, silty to pure limestone, siltstone and shale at base	
	Champlainian	Galena Group	Galena-Platteville dolomites	Cambrian-Ordovician Aquifer		
		Platteville Group	Galena-Platteville dolomites	Cambrian-Ordovician Aquifer	Dolomite and limestone, locally cherty, sandy at base, shale partings	

TABLE 2.4-20 (Cont'd)

SYSTEM	SERIES	GROUP OR FORMATION	HYDROGEOLOGIC UNIT		DESCRIPTION	HYDROGEOLOGIC CHARACTERISTICS
		Ancell Group	Glenwood-St. Peter sandstone	Cambrian-Ordovician Aquifer	Sandstone, shale at top, little dolomite, locally cherty at base	Groundwater occurs under leaky artesian conditions in the sandstones and in joints in the dolomites. Yields are variable and depend upon which units are open to the well.
	Canadian	Prairie du Chien Group	Prairie du Chien, Eminence, Potosi and Franconia dolomites	Cambrian-Ordovician Aquifer	Sandy dolomite, dolomitic sandstone, cherty at top, interbedded shale in lower part	In terms of the total yield of a well penetrating the entire thickness of the Cambrian-Ordovician Aquifer, the Glenwood-St. Peter sandstone supplies about 15 percent, the Prairie du Chien, Eminence, Potossi and Franconia dolomites collectively supply about 35 percent, and the Ironton-Galesville sandstone supplies about 50 percent.
CAMBRIAN	Croixan	Eminence Formation		Cambrian-Ordovician Aquifer		
		Potosi Dolomite		Cambrian-Ordovician Aquifer		
		Franconia Formation		Cambrian-Ordovician Aquifer		

TABLE 2.4-20 (Cont'd)

SYSTEM	SERIES	GROUP OR FORMATION	HYDROGEOLOGIC UNIT		DESCRIPTION	HYDROGEOLOGIC CHARACTERISTICS
		Ironton Sandstone	Ironton-Galesville sandstone	Cambrian-Ordovician Aquifer	Sandstone, upper part dolomite	
		Galesville Sandstone		Cambrian-Ordovician Aquifer		
		Eau Claire Formation	Eau Claire shale (upper and middle beds)	Aquitard	Shales, dolomites and shaly dolomitic sandstone	Insignificant amounts of ground water may occur in joints. These beds act as a confining layer between the Cambrian-Ordovician Aquifer and the Mt. Simon Aquifer.
		Mt. Simon Sandstone	Eau Claire and Mt. Simon sandstones	Mt. Simon Aquifer	Sandstone	Groundwater occurs under leaky artesian conditions. Groundwater in this aquifer is too highly mineralized for most purposes. Adequate supplies for municipal and industrial use are more easily obtained from shallower aquifers.

TABLE 2.4-21

QUALITY OF GROUNDWATER IN THE GLACIAL DRIFT

PARAMETER*	MAXIMUM CONCENTRATION	MINIMUM CONCENTRATION	AVERAGE CONCENTRATION**
pH	8.5	7.3	7.8
Arsenic (total)	0.036	0.001	0.005
Boron (soluble)	1.7	0.2	0.2
Calcium (soluble)	60	21	38
Chloride	6	0.02	2.7
Iron (soluble)	1.11	0.02	0.11
Iron (total)	24.0	0.04	1.2
Magnesium (soluble)	19	7	13
Sulfate	80	13	36
Total alkalinity (as $CaCO_3$)	176	52	106
Total dissolved solids	296	106	192
Total hardness (as $CaCO_3$)	218	80	146
Total suspended solids	457	2	43

Note: Samples were collected from each of eight observation wells around the main plant excavation beginning January 15, 1976. The locations of the observation wells are shown on Figure 2.4-36. Installation details of a typical observation well are shown on Figure 2.4-43.

* All parameters except pH are reported in mg/l.

** Values represent an average of 15 tests from each observation well.

TABLE 2.4-22A

ISGS PUBLIC GROUNDWATER SUPPLIES WITHIN 10 MILES

	COORDINATES												PUMPING INFORMATION			
ISWS Well ID	LATITUDE	LONGITUDE	TOTAL DEPTH (FEET BGS[1])	SECTION	TOWNSHIP	RANGE	GROUND SURFACE ELEVATION (FEET AMSL[2])	COMPLETION DATE (YYYYMMDD)	WELL LOCATION/OWNER	WELL NUMBER	INSTALLED BY	WELL TYPE	STATIC WATER LEVEL (FT BTOC[3])	PUMPING LEVEL (FT BTOC)	PUMPING RATE (GPM[4])	HOURS PUMPED
120630090800	41.387769	-88.269681	1499	35	34	8	510	19570101	Dresden Nuc Pow Sta	1	WehlingWellWorksInc.					
120630134100	41.398263	-88.321828	1490	28	34	8	501	19620101	Desplaines Chem Co		Miller,J.P.Art.Well					
120630142900	41.455381	-88.258803	1508	1	34	8	611	19650501	City Of Minooka	3	WehlingWellWorksInc.					
120630146300	41.414021	-88.316296	1540	21	34	8	531	19670901	Amax Aluminum Co		WehlingWellWorksInc.					
120630152000	41.415059	-88.34037	1453	20	34	8	530	19681101	Northern Petro Co	1	LayneWesternCo.,Inc.					
120630201500	41.410622	-88.342306	1858	20	34	8	526	19691031		M-69-3	McCarthyEngr.&Const.					
120630203700	41.408057	-88.328204	1470	21	34	8	523	19701001	Northern Petro Co	2	LayneWesternCo.,Inc.					
120630205100	41.398546	-88.318585	1455	28	34	8	500	19701201	Northern Petro Co	5	LayneWesternCo.,Inc.					
120630205200	41.403426	-88.328151	1463	21	34	8	517	19700101	Northern Petro Co	3	LayneWesternCo.,Inc.					
120630210800	41.413055	-88.316284	1515	21	34	8	527	19711101	Amax Aluminum Co	2	WehlingWellWorksInc.					
120630218500	41.41144	-88.307107	1511	22	34	8	523	19721201	N Ill Gas Co	1	LayneWesternCo.,Inc.					
120630219600	41.411575	-88.301783	1519	22	34	8	530	19730401	N Ill Gas Co	2	LayneWesternCo.,Inc					
120632306600	41.412982	-88.317494	1540	21	34	8	0	19890714	Alumax Mill Products	3	Wehling,RichardH.	COMM	335	0	0	0
120632411200	41.395053	-88.308601	1490	28	34	8	0		Des Plaines Chem. Co	2	Miller,J.P.DrlgCo.	PRIV	280	400	30	1
120632442000	41.434067	-88.269398	1601	12	34	8	605	20050609	Minooka, Village of	9	WaterWellSolutions	MUNIC	427	612	1500	24
121970025100	41.443508	-88.160268	1407	11	34	9	540	19551001	Stephan Chemical	1	LayneWestern		0	0	0	0
121970033100	41.447404	-88.172982	1420	11	34	9	571	19571001	Amoco Chem Corp	1	LayneWestern		0	0	0	0
121970054200	41.448429	-88.17618	1405	10	34	9	573	19580501	Amoco Chemical	2	LayneWestern		0	0	0	0
121970060300	41.392437	-88.145771	1627	25	34	9	606	19410101	Kankakee Ordn Works	8	LayneWestern		0	0	0	0
121970060400	41.39932	-88.146048	1603	25	34	9	589	19410101	Kankakee Ordn Works	9	LayneWestern		0	0	0	0
121970060500	41.406192	-88.146326	1571	25	34	9	591	19411001	Kankakee Ordn Works	10	LayneWestern		0	0	0	0
121970060600	41.377243	-88.180683	1593	34	34	9	528	19410101	Kankakee Ordn Works	3	LayneWestern		0	0	0	0
121970060700	41.377124	-88.187975	1554	34	34	9	522	19410101	Kankakee Ordn Works	4	LayneWestern		0	0	0	0
121970060800	41.377367	-88.164341	1598	35	34	9	539	19410301	Kankakee Ordn Works	1	LayneWestern		0	0	0	0
121970060900	41.377254	-88.173364	1605	35	34	9	532	19410101	Kankakee Ordn Works	2	LayneWestern		0	0	0	0
121970061100	41.378701	-88.144999	1653	36	34	9	577	19410801	Kankakee Ordn Works	6	LayneWestern		0	0	0	0
121970061200	41.385583	-88.145393	1655	36	34	9	601	19410101	Kankakee Ordn Works	7	LayneWestern		0	0	0	0
121970121200	41.44623	-88.161567	1402	11	34	9	530	19601201	Stephan Chemical	2	LayneWestern		0	0	0	0
121970132800	41.444811	-88.174458	1400	11	34	9	572	19640601	Amoco Chemicals	3	MilaegerWellDrlg.		0	0	0	0
121970181500	41.407579	-88.212374	1555	21	34	9	520	19700201	Durkee Glidden	1	WehlingWellWorksInc.		0	0	0	0
121970184500	41.412387	-88.188452	1578	22	34	9	556	19700401	Mobil Oil Co	1	WehlingWellWorks,Inc.		0	0	0	0
121970274000	41.383657	-88.188285	1593	34	34	9	530	19720801	Chicago-Joliet Livestock Market	2	WehlingWellWorks,Inc.		0	0	0	0
121972408000	41.44303	-88.160164	1410	11	34	9	520	19730501	Stephan Chemical	3	LayneWestern		0	0	0	0
121972415500	41.403716	-88.203386	1605	28	34	9	535	19740201	Dow Chem Usa	1	WehlingWellWorksInc.					
121972559100	41.45126	-88.177639	1415	3	34	9	570	19751201	Amoco Chemical	4	WehlingWellWorks,Inc.		0	0	0	0
121972609600	41.406758	-88.212144	1555	21	34	9	0	19770301	Durkee Glidden	2	Wehling,R.H.					
121973287700	41.417823	-88.196939	1573	21	34	9	545	19650901	Rexall Chem. Co.	1	WehlingWellWorks,Inc.		191	588	950	0
121973676700	41.395537	-88.241901	1647	30	34	9	603	19930526	Channahon, Village of	4	Wehling,RichardH.	MUNIC	395	700	1000	24
121973728100	41.449314	-88.159436	1415	11	34	9	0	19960724	Stephan Chemical	4	Buffington,G.	INDUS	615	751	600	24

TABLE 2.4-22A

ISGS PUBLIC GROUNDWATER SUPPLIES WITHIN 10 MILES (Continued)

	COORDINATES		TOTAL DEPTH				GROUND SURFACE ELEVATION	COMPLETION DATE				WELL	STATIC WATER LEVEL	PUMPING INFORMATION		
ISWS Well ID	LATITUDE	LONGITUDE	(FEET BGS[1])	SECTION	TOWNSHIP	RANGE	(FEET AMSL[2])	(YYYYMMDD)	WELL LOCATION/OWNER	WELL NUMBER	INSTALLED BY	TYPE	(FT BTOC[3])	PUMPING LEVEL (FT BTOC)	PUMPING RATE (GPM[4])	HOURS PUMPED
121973946200	41.406758	-88.212144	1560	21	34	9	0		Glidden Durkee	2	Wehling,RichardH.		0	0	0	0
121973994100	41.458764	-88.184041	1460	3	34	9	0	20000413	Joliet, City of	18	Layne-WesternCo.	NCPUB	585	747	1000	26
121974066500	41.416073	-88.183257	1597	22	34	9	0	20021226	Exxon Mobil Refinery	2	Layne-WesternCo.	NPOT	362	564	1241	0
121974157400	41.41612	-88.181765	1595	22	34	9	0	20050617	Exxon Mobil Oil Corp.	3	Buffington,G.	COMM	460	625	1209	0
121974203500	41.404002	-88.241851	1625	30	34	9	606	20060317	Channahon, Village of	6	Layne-WesternCo.	MUNIC	425	597	1102	24
121974206600	41.445837	-88.180758	1490	10	34	9		20061103	Flint Hills Resources, LP	5	WaterWellSolutions		577	955	450	3
121974240400	41.419777	-88.19351	1589	22	34	9	550	20070817	Exxon Mobil Oil Corp.	4	Layne-WesternCo.	NPOT	465	783	1445	24
120630107900	41.358215	-88.425045	1485	4	33	7	523	19150101	Morris City	3						
120630108000	41.361798	-88.429821	1462	4	33	7	505	19540101	Morris City	5	MilaegerWellDrlg.					
120630109500	41.35672	-88.427444	1501	9	33	7	510	19380101	Morris City	4	Mileage&Smyth					
120630222400	41.348741	-88.352708	1513	7	33	8	526	19730401	Commonwealth Edison	3	WehlingWellWorksInc.					
120630222500	41.350529	-88.352905	1477	7	33	8	525	19730301	Commonwealth Edison	2	WehlingWellWorksInc.					
120630222600	41.353539	-88.354513	1510	7	33	8	515	19730301	Commonwealth Edison	1	WehlingWellWorksInc.					
120630222700	41.350547	-88.362874	1495	7	33	8	505	19730501	Commonwealth Edison	4	WehlingWellWorksInc.					
120632236600	41.293501	-88.286723	1785	34	33	8	565	19780601	Coal City	5	LayneWesternCo.,Inc.					
120632335500	41.361685	-88.437293	1450	4	33	7	0	19931031	Morris, City of	6	Albrecht,S.Dean	TH	194	389	1600	15
120632429800	41.358226	-88.425053	1449	4	33	7	0	20020610	Morris, City of	7	MeadowEquipment	MUNIC	209	0	0	0
120632444600	41.355691	-88.458835	1375	8	33	7	0	20050711	Exelon Midwest Fire Trainin	2	Kerry,CharlesM.	NCPUB	267	440	240	24
121970059100	41.360813	-88.076862	1672	9	33	10	647		Elwood Ordnance		Miller,J.P.Art.Well		0	0	0	0
121970059200	41.364557	-88.084262	1645	9	33	10	640	19410101	Elwood Ordnance	2	Miller,J.P.Art.Well		0	0	0	0
121970068000	41.36084	-88.135488	1635	12	33	9	575	19420101	Kankakee Ordnance Works	11	LayneWestern		0	0	0	0
121970071500	41.308272	-88.146212	1565	25	33	9	540	19360101	Wilmington City	2	VarnerCW		0	0	0	0
121970133100	41.304774	-88.1471	1575	36	33	9	530	19641101	Wilmington City of	3	WehlingWellWorks,Inc.		0	0	0	0
121970001000	41.263928	-88.218262	1647	8	32	9	577	19370101	Braidwood City	1	VarnerCW					
121972484600	41.244594	-88.230882	1760	19	32	9	597	19741001	Commonwealth Edison	1	WehlingWellWorks,Inc.		0	0	0	0
121972722600	41.2803	-88.220957	1732	5	32	9	0	19790731	Braidwood, City of	3	Wehling,RichardH.	MUNIC	237	415	1250	8
121973118900	41.222538	-88.187879	1690	28	32	9	0	19811029	Commonwealth Edison Co.		Wehling,WendellE.	COMM	0	0	0	0
121973708700	41.212446	-88.025161	1700	36	32	10	603	19860826	Capital Development Board		Wehling,WendellE.	NCPUB	0	0	0	0
121974267700	41.244624	-88.227214	1750	19	32	9		20090422	Exelon-Braidwood		Wehling,RichardW.	NPOT	288	335	610	24
120630151600	41.186958	-88.301279	1933	4	31	8	587	19681001	Vlge Of Gardner	4	LayneWesternCo.,Inc.					
120632333600	41.187676	-88.305034	1929	4	31	8	0	19930618	Village of Gardner #5		Wehling,RichardH.	MUNIC	219	281	0	8
120632411300	41.172875	-88.275535	1220	11	31	8	0	20010109	South Wilmington	5	RichardW.Wehling	NCPUB	232	417	0	8

Note:

[1] BGS - below ground surface

[2] AMSL - above mean sea level

[3] BTOC - below top of casing

[4] GPM - gallons per minute

All townships and ranges within the 10-mile radius were included within the search.

Therefore, some wells listed may be outside of the 10-mile radius.

TABLE 2.4-22B

ISWS PUBLIC GROUNDWATER SUPPLIES WITHIN 10 MILES

	COORDINATES											
ISWS WELL ID	LATITUDE	LONGITUDE	TOTAL DEPTH (FEET BGS[1])	SECTION	TOWNSHIP	RANGE	PLOT	GROUND SURFACE ELEVATION (FEET AMSL[2])	COMPLETION DATE (YYYYMMDD)	WELL LOCATION/OWNER	INSTALLED BY	WELL USE
405925	41.1891667	88.3077778	1933	04	31N	08E	1A	588	1968	GARDNER	LAYNE-WESTERN CO	
410240	41.1877778	88.3055556	1929	04	31N	08E	2A	587	1993	GARDNER	L-W CO-WEHLING DIV	
400036	41.1728750	88.2755350	1220	11	31N	08E	6A		20010109	SOUTH WILMINGTON	RICHARD W. WEHLING	CS
405968	41.3577778	88.4258333	1485	04	33N	07E	2A	522	1915	MORRIS	LAYNE-WESTERN	CS
405969	41.3625000	88.4297222	1462	04	33N	07E	4C	506	1954	MORRIS	MILAEGER WELL & PUMP	CS
410360	41.3622222	88.4361111	1451	04	33N	07E	6C	525	1993	MORRIS	ALBRECHT WELL DRLG	CS
412026			1375	08	33N	07E	8H		2005	EXELON MIDWEST FIRE TRAINING ACADEMY	MEADOW EQUIPMENT	IC
410959	41.3569444	88.4305556	1449	09	33N	07E	4H	510	2002	MORRIS	MEADOW EQUIPMENT	CS
405976	41.2936111	88.2869444	1785	34	33N	08E	1D	560	1978	COAL CITY	LAYNE-WESTERN CO	
405974			1788	34	33N	08E	6F	560	1893	CARBON HILL	--	
405989	41.4561111	88.2525000	1508	01	34N	08E	3E	610	1965	MINOOKA	WEHLING WELL WORKS	CS
411799			1601	12	34N	08E	8A	605	20050609	MINOOKA	WATER WELL SOLUTIONS DRILLING DIVISION	CS
401644			1453	20	34N	08E	2E		1968	LYONDELL/EQUISTAR - MORRIS	LAYNE-WESTERN	IC
429358			1470	20	34N	08E	3C			AKZO NOBEL SURFACE CHEMISTRY		IC
401623			1515	21	34N	08E	3F	528	1971	SAPA INDUSTRIAL EXTRUSIONS	E.C. WEHLING	IC
401621			1540	21	34N	08E	3G	530	1967	SAPA INDUSTRIAL EXTRUSIONS		IC
401624			1540	21	34N	08E	4F	528	1989	SAPA INDUSTRIAL EXTRUSIONS		IC
401647			1463	21	34N	08E	8A		1971	LYONDELL/EQUISTAR - MORRIS	LAYNE-WESTERN	IC
401646			1470	21	34N	08E	8C		1970	LYONDELL/EQUISTAR - MORRIS	LAYNE-WESTERN	IC
401654			1519	22	34N	08E	6E		19720000	NORTH IL GAS SNG PLANT		IC
401653			1511	22	34N	08E	8E		19720000	NORTH IL GAS SNG PLANT		IC
401649			1492	28	34N	08E	1D	490		LYONDELL/EQUISTAR - MORRIS		IC
401648			1455	28	34N	08E	5F		1970	LYONDELL/EQUISTAR - MORRIS	LAYNE-WESTERN	IC
401674			1500	35	34N	08E	1E	510	1957	EXELON - DRESDEN STATION		IC
401673			1499	35	34N	08E	1G	515	1957	EXELON - DRESDEN STATION		IC
56459			1906	20	34N	08E			19780000	U P G INC	RECKTOR AND STONE	IC
56460			1925	20	34N	08E			19780000	U P G INC	RECTOR AND STONE	IC
56461			1925	20	34N	08E			19780000	U P G INC	RECKOR AND STONE	IC
56472			1492	28	34N	08E	1D		19620509	DES PLAINES CHEMICAL CO	ARTESIAN WELL CO(MIL	IC
56514			1500	35	34N	08E	5A		19570000	DRESDEN NUC POWER STAT	WHELING	IC
72629			1260	28	30N	10E	8D		00000000	TEXAS ILLINOIS	EARLE & HOLDER	IC
72630			1883	28	30N	10E	8D		19510400	TEXAS ILLINOIS	EARLE & HOLDER	IC
72633			1900	31	30N	10E	7G		00000000	NATURAL GAS STORAGE WELL		IC
72640			1404	32	30N	10E	2H		19510500	TEXAS ILLINEOS NATRUAL GAS	EAKLE & HOLDER	IC
72641			1861	32	30N	10E	2H		19510000	NATURAL GAS STORAGE CO. OF ILL	EAKLE & HOLDER	IC
72642			1710	32	30N	10E	2H		19510616	NATURAL GAS STORAGE CO. OF ILL	EAKLE & HOLDER	IC
74788			2196	03	30N	09E	8F		19650000	NAT GAS PIPELINE CO		IC
74787			1672	03	30N	09E	8G		19650000	NAT GAS PIPELINE CO		IC
409163	41.2805556	88.2213889	1733	05	32N	09E	6D	560	1979	BRAIDWOOD	WEHLING WELL WORKS	
409164	41.2641667	88.2191667	1647	08	32N	09E	5C	575	1937	BRAIDWOOD	C VARNER/W L THORNE	
409168			1700	36	32N	10E	2D	610	1986	KANKAKEE CORRECTIONAL CENTER	WEHLING WELL WORKS	ST
160754			1700	36	32N	10E	2D		19680800	ILL YTH CENTER	WEHLING	PK

TABLE 2.4-22B

ISWS PUBLIC GROUNDWATER SUPPLIES WITHIN 10 MILES (Continued)

ISWS WELL ID	COORDINATES LATITUDE	COORDINATES LONGITUDE	TOTAL DEPTH (FEET BGS[1])	SECTION	TOWNSHIP	RANGE	PLOT	GROUND SURFACE ELEVATION (FEET AMSL[2])	COMPLETION DATE (YYYYMMDD)	WELL LOCATION/OWNER	INSTALLED BY	WELL USE
409784			1652	01	33N	09E	5D	560	1942	MIDEWIN NATIONAL TALLGRASS PRAIRIE - USDA FS	VARNER WELL & PUMP	IC
409788			1644	12	33N	09E	1G	575	1942	MIDEWIN NATIONAL TALLGRASS PRAIRIE - USDA FS	LAYNE-WESTERN CO	IC
409173	41.3041667	88.1472222	1578	36	33N	09E	7H	530	1964	WILMINGTON	WEHLING WELL WORKS	CS
149179			1653	01	33N	09E			19420114	KANKEE ORDANCE WORKS	VARNER	IC
149217			1630	12	33N	09E	1G		19440000	US RUBBER CO 11		IC
409811	41.3593611	88.0771111	1672	09	33N	10E	1F	647	1941	MIDEWIN NATIONAL TALLGRASS PRAIRIE - USDA FS	J P MILLER ART WELL	IC
409783	41.3631944	88.0843611	1645	09	33N	10E	4H	640	1941	MIDEWIN NATIONAL TALLGRASS PRAIRIE - USDA FS	J P MILLER ART WELL	IC
149453			1672	09	33N	10E			19410000	ELWOOD ORDANCE PLANT		IC
149454			1645	09	33N	10E			19410000	ELWOOD ORD PLANT		IC
404053			1415	03	34N	09E	1A	570	19751231	FLINT HILLS RESOURCES	MILAEGER WELL & PUMP CO.	IC
400038			1460	03	34N	09E	4F		20000413	JOLIET	LAYNE-WESTERN	CS
404050			1405	10	34N	09E	1H	573	19580500	FLINT HILLS RESOURCES	LAYNE WESTERN CO.	IC
423302	41.4452444	88.1844708	1515	10	34N	09E	4F	576	20061103	FLINT HILLS RESOURCES	WATER WELL SOLUTIONS	IC
405373			1415	11	34N	09E	2A	535	1996	STEPAN CHEMICAL COMPANY	LAYNE-WESTERN CO	IC
404032			1410	11	34N	09E	2E	525	1973	STEPAN CHEMICAL COMPANY	LAYNE-WESTERN CO	IC
404031			1402	11	34N	09E	3D	520	19601209	STEPAN CHEMICAL COMPANY	LAYNE-WESTERN CO	IC
404049			1422	11	34N	09E	7G	571	19571022	FLINT HILLS RESOURCES	LAYNE-WESTERN COMPANY	IC
404052	41.4448110	88.1744580	1369	11	34N	09E	8F	570	19640601	FLINT HILLS RESOURCES	MILAEGER WELL & PUMP CO.	IC
404080			1573	21	34N	09E	2D		19650823	INEOS NOVA LLC	WEHLING WELL WORKS, INC.	IC
411911			1595	22	34N	09E	4F	562	2005	EXXON MOBIL OIL CORP - JOLIET REFINERY	LAYNE-WESTERN (AURORA)	IC
411133			1597	22	34N	09E	4F	545	2002	EXXON MOBIL OIL CORP - JOLIET REFINERY	LAYNE-WESTERN	IC
432165	41.3953333	89.2461167	1589	22	34N	09E	6F	550	20071024	EXXON MOBIL OIL CORP - JOLIET REFINERY	LAYNE-WESTERN	IC
404067			1578	22	34N	09E	7D	556	1970	EXXON MOBIL OIL CORP - JOLIET REFINERY	WEHLING	IC
409821			1627	25	34N	09E	5A	606	1941	ELWOOD	LAYNE-WESTERN CO	CS
409790			1603	25	34N	09E	5D	590	1941	ELWOOD	LAYNE-WESTERN CO	CS
409789			1571	25	34N	09E	5H	591	1941	ELWOOD	LAYNE-WESTERN CO	CS
404008			1605	28	34N	09E	5H	534	1974	DOW CHEMICAL	WEHLING WELL WORKS	
411969	41.4059167	88.2419167	1625	30	34N	09E	4H	606	20060317	CHANNAHON	LAYNE-WESTERN	CS
410382	41.3863889	88.2290278	1647	30	34N	09E	5D	603	1993	CHANNAHON	L-W CO-WEHLING DIV	CS
409814			1593	34	34N	09E	3A	528	1941	MIDEWIN NATIONAL TALLGRASS PRAIRIE - USDA FS	LAYNE-WESTERN CO	IC
409815			1551	34	34N	09E	7A	522	1941	MIDEWIN NATIONAL TALLGRASS PRAIRIE - USDA FS	LAYNE-WESTERN CO	IC
404027			1593	34	34N	09E	7D	530	1972	HAGER'S COUNTRY MARKET, INC	WEHLING WELL WORKS	

TABLE 2.4-22B

ISWS PUBLIC GROUNDWATER SUPPLIES WITHIN 10 MILES (Continued)

	COORDINATES											
ISWS WELL ID	**LATITUDE**	**LONGITUDE**	**TOTAL DEPTH (FEET BGS[1])**	**SECTION**	**TOWNSHIP**	**RANGE**	**PLOT**	**GROUND SURFACE ELEVATION (FEET AMSL[2])**	**COMPLETION DATE (YYYYMMDD)**	**WELL LOCATION/OWNER**	**INSTALLED BY**	**WELL USE**
409812			1597	35	34N	09E	5A	539	1941	MIDEWIN NATIONAL TALLGRASS PRAIRIE - USDA FS	LAYNE-WESTERN CO	IC
409813			1612	35	34N	09E	8A	532	1941	MIDEWIN NATIONAL TALLGRASS PRAIRIE - USDA FS	LAYNE-WESTERN CO	IC
409817			1653	36	34N	09E	5A	578	1941	ELWOOD	LAYNE-WESTERN CO	CS
409818	42.3916667	88.1416667	1655	36	34N	09E	5E	601	1941	ELWOOD	LAYNE-WESTERN CO	CS
150362			1415	03	34N	09E	1A		19751231	AMACO CHEM CO	WHELING	IC
150302			1375	11	34N	09E	8H		19571100	AMOCO CHEMICAL CO		IC
150310			1407	14	34N	09E			19551000	STEPAN CHEM CO	LAYNE WESTERN	IC
150785			1555	21	34N	09E	8B		19770300	LODERS CROKLAAN	WEHLING	IC
294024			1647	30	34N	09E	5D		19930700	VILLAGE OF CHANNAHON #4	LAYNE-WESTERN	CS
150808			1648	36	34N	09E	5A		19420000	US RUBBER UNIROYAL CO	JP MILLER	IC
444377			1600	07	34N	10E				JOLIET		CS
411507			1581	07	34N	10E	5A			ELWOOD ENERGY LLC		IC
405294	41.4155556	88.1066667	1725	20	34N	10E	4E	640	1996	ELWOOD	LAYNE-WESTERN CO	CS
409787			1709	31	34N	10E	7A	626	1943	MIDEWIN NATIONAL TALLGRASS PRAIRIE - USDA FS	LAYNE-WESTERN CO	IC
311458			1665	30	34N	10E	6A		19990430	VETERANS ADM.-CHG.NATL.CEMETRY	LAYNE-WESTERN/HALL	NC
150842			1698	31	34N	10E			19430100	KANKAKEE ORDANCE WORKS	LAYNE WESTERN	IC
412019			1730	09	34N	11E	7A	690	2006	MANHATTAN	LAYNE-WESTERN	CS
410378	41.4472222	87.9975000	1703	17	34N	11E	5D	685	1993	MANHATTAN	L-W CO-WEHLING DIV	CS
400177			1770	21	34N	11E	5F		2000	MANHATTAN	LAYNE-WESTERN	CS

Note: [1] BGS - below ground surface

[2] AMSL - above mean sea level

Well use:
CS - Community Supply
DO - Domestic
IC - Industrial/Commercial
IR - Irrigation
NC - Non-Community Supply
NR - Not Reported
PK - Park
ST - State

All townships and ranges within the 10-mile radius were included within the search. Therefore, some wells listed may be outside of the 10-mile radius.

TABLE 2.4-22C

ISGS PUBLIC GROUNDWATER SUPPLIES WITHIN 25 MILES

	COORDINATES		TOTAL DEPTH				GROUND SURFACE ELEVATION	COMPLETION DATE				WELL	STATIC WATER LEVEL	PUMPING INFORMATION		
ISWS Well ID	LATITUDE	LONGITUDE	(FEET BGS[1])	SECTION	TOWNSHIP	RANGE	(FEET AMSL[2])	(YYYYMMDD)	WELL LOCATION/ OWNER	WELL NUMBER	INSTALLED BY	TYPE	(FT BTOC[3])	PUMPING LEVEL (FT BTOC)	PUMPING RATE (GPM[4])	HOURS PUMPED
120932380900	41.566368	-88.274852	1555	25	36	8	665	20030403	Joliet, City of	21	Buffington, G.	MUNIC	605	848	976	0
120932416900	41.551969	-88.277415	1556	35	36	8	660	20030504	Joliet, City of	20D	Buffington, G.	MUNIC	604	806	1044	24
121970053700	41.564509	-88.062438	1588	35	36	10	633	19580101	Fairmt Schl Dist 89	3	Wehling Well Works Inc.		0	0	0	0
121970127900	41.603715	-88.093666	1523	16	36	10	668		Globe Aircraft	3	Miller, J. P. Art. Well		0	0	0	0
121970128000	41.589284	-88.061894	1922	23	36	10	568		Lockport City	1	Miller, J. P. Art. Well		0	0	0	0
121970128100	41.588843	-88.058411	1475	23	36	10	582	19270101	Lockport City	2	Miller, J.P. Artesian Well Co.		0	0	0	0
121970128200	41.582798	-88.055087	1571	23	36	10	662	19400701	Lockport City	3	Miller, J.P. Artesian Well Co.		0	0	0	0
121970128300	41.592149	-88.049273	1572	23	36	10	648	19540101	Lockport City	4	Miller, J.P. Artesian Well Co.		0	0	0	0
121970128400	41.573559	-88.054673	1605	26	36	10	631	19430101	Carnegie-Ill Steel		Miller, J. P. Art. Well		0	0	0	0
121970128600	41.578616	-88.096859	1600	28	36	10	0	19200101	Ill State Prison	1	Miller, J. P. Art. Well		0	0	0	0
121970128700	41.57724	-88.096829	1577	28	36	10	640	19210101	Ill State Prison	2	Miller, J. P. Art. Well		0	0	0	0
121970128800	41.579989	-88.097077	1527	28	36	10	650	19260901	Ill State Prison	3	Gray Well Drilling		0	0	0	0
121970128900	41.576681	-88.097182	2007	28	36	10	640	19370101	Ill State Prison	4	Thorne, W. L. Co.		0	0	0	0
121970129100	41.577819	-88.117919	1653	29	36	10	645	19510101	Stateville Prison	5	Miller, J. P. Art. Well		0	0	0	0
121970129300	41.552538	-88.102075	1652	32	36	10	656	19450901	Lidice City	3	Miller, J. P. Art. Well		0	0	0	0
121970129500	41.566307	-88.095104	1558	33	36	10	596	19320101	Public Serv Co		Miller, J. P. Art. Well		0	0	0	0
121970129700	41.560927	-88.074301	2076	34	36	10	552	19960101	Joliet Steel Mill		Wallen A K		0	0	0	0
121970133400	41.582313	-88.0914	1611	21	36	10	642	19660101	Stateville Penitentiary	6	Wehling Well Works, Inc.		0	0	0	0
121972552600	41.572088	-88.156744	1557	25	36	9	600	19760501	Joliet City Of	12D	Wehling Well Works, Inc.		0	0	0	0
121973677300	41.594346	-88.204602	1508	16	36	9	604	19910424	Plainfield, Village of	5	Wehling, Richard H.	MUNIC	0	0	0	0
121973843100	41.562769	-88.236416	1566	32	36	9	0	19970917	Joliet, City of	15	Layne-Western Co.		570	772	1100	12
121973909500	41.588899	-88.056001	1475	23	36	10	0	20000621	Prairie Bluff Golf Club	2	Robinson Engineering		0	0	0	0
121973989900	41.60242	-88.092719	1523	16	36	10	0		Lewis University			BUS	0	0	0	0
121974003600	41.553697	-88.076562	1547	34	36	10	0		Joliet Crctl Ctr			NCPUB	0	0	0	0
121974107000	41.55036	-88.249759	1525	31	36	9	0	20000616	Joliet, City of	17	Layne-Western Co.	MUNIC	0	0	0	0
121970129900	41.556031	-88.023292	1656	31	36	11	642	19500101	Joliet City Of	3	Miller, J. P. Art. Well		0	0	0	0
120932466600	41.534722	-88.269444	1523	12	35	7	636	20060504	Joliet, City of	27D	Layne-Western Co.	MUNIC	593	951	1119	25
120932442600	41.482077	-88.279394	1520	26	35	8	646	20050208	Minooka, Village of	8	Water Well Solutions	MUNIC	433	763	1500	24
120932460200	41.531942	-88.282922	1533	11	35	8	0	20050913	Joliet, City of	25D	Hall, Edward - Web Well & Pump	MUNIC	541	967	671	24
120932468400	41.549722	-88.265	1554	1	35	8		20060323	Joliet, City of	28D	Layne-Western Co.	NCPUB	140	441	50	2
121970026600	41.536733	-88.185097	1520	3	35	9	592	19570901	Dupage River Farm		Miller, J. P. Art. Well		0	0	0	0
121970119800	41.486607	-88.139142	1555	25	35	9	545	19611201	Caterpillar Tactor Co.	3	Miller Well Co		0	0	0	0
121970123200	41.522909	-88.185469	1350	10	35	9	575	19620101	Holiday Motel	1	Miller, J. P. Art. Well		0	0	0	0
121970133200	41.524293	-88.184955	1449	10	35	9	580	19640801	Holiday Inn	2	Miller, J. P. Art. Well		0	0	0	0
121972407900	41.477018	-88.206043	1435	33	35	9	575	19731001	Camelot Utility Co	1	K & K Well Drilling		0	0	0	0
121972538500	41.522291	-88.197632	1440	10	35	9	575	19760101	Dworkin Allan		K & K Well Drilling		0	0	0	0
121972538501	41.522291	-88.197632	1455	10	35	9	0	19900203	Imperial Mobile Home Park	3	Fykes, Charles N.	NCPUB	540	620	0	1
121972552700	41.544038	-88.146968	1623	1	35	9	619	19760301	Joliet City Of	11D			0	0	0	0
121973322000	41.522909	-88.185447	1458	10	35	9	0		Holiday Motel	1	Miller, J.P. Co.	BUS	333	345	45	4
121973360900	41.525454	-88.204095	1490	9	35	9	0		Will Co. Water Co. (Shorewood)		Neely, Larry C.		0	0	0	0
121973912300	41.524531	-88.161333	1572	11	35	9	0	19700101	Joliet, City o	10-D		MUNIC	0	0	0	0

TABLE 2.4-22C

ISGS PUBLIC GROUNDWATER SUPPLIES WITHIN 25 MILES (Continued)

	COORDINATES		TOTAL DEPTH				GROUND SURFACE ELEVATION	COMPLETION DATE					STATIC WATER LEVEL	PUMPING INFORMATION		
ISWS Well ID	LATITUDE	LONGITUDE	(FEET BGS[1])	SECTION	TOWNSHIP	RANGE	(FEET AMSL[2])	(YYYYMMDD)	WELL LOCATION/ OWNER	WELL NUMBER	INSTALLED BY	WELL TYPE	(FT BTOC[3])	PUMPING LEVEL (FT BTOC)	PUMPING RATE (GPM[4])	HOURS PUMPED
121973982500	41.547836	-88.1912	1530	3	35	9	0	19990101	Joliet, City of	16		MUNIC	0	0	0	0
121974164200	41.523474	-88.141468	1618	12	35	9	0	20050114	Joliet	22D	Layne-Western Co.	UNIC	820	970	708	24
121974247000	41.49212	-88.246346	1631	19	35	9		20071106	Shorewood, Village of	8	Water Well Solutions	UNIC	440	820	1507	24
121974260100	41.49142	-88.225054	1631	29	35	9		20080130	Shorewood, Village of	9	Water Well Solutions	UNIC	641	896	1507	24
121970026500	41.494546	-88.113188	1500	20	35	10	530	19580101	Commonwealth Edison Station #9	2	Milaeger Well Drlg.		0	0	0	0
121970056100	41.499088	-88.089256	1604	21	35	10	585		Amer Cyanamid	1	Miller, J. P. Art. Well		0	0	0	0
121970056300	41.492296	-88.12122	1535	30	35	10	0	19520101	Blockson Chem	5	Layne Western		0	0	0	0
121970063300	41.541484	-88.061803	1609	2	35	10	555	19240101	Joliet City		Sewell Well Co		0	0	0	0
121970063400	41.545945	-88.074867	1550	3	35	10	555	19280101	Ill St Penitentary	1	Gray Well Drilling		0	0	0	0
121970063500	41.546304	-88.071574	1600	3	35	10	550	19480101	Ill St Penitentary	3	Miller J P Well Co.		0	0	0	0
121970063600	41.54901	-88.084588	1596	4	35	10	558	19240101	Calumet Chem		Cater William		0	0	0	0
121970063900	41.530775	-88.085666	1570	9	35	10	532		Joliet City Of		Ohio Drlg Co		0	0	0	0
121970064000	41.52984	-88.083561	1621	9	35	10	543	19070101	Joliet City Of	3	Wilson L					
121970064100	41.536534	-88.083947	1568	9	35	10	545	19170101	Joliet City Of		Ohio Drlg Co					
121970064300	41.528962	-88.072138	1460	10	35	10	550	19380101	Beatrice Creamery		Sewell Well Co					
121970064400	41.526495	-88.071775	1483	10	35	10	545	19380101	Joliet Citizens Brew		Sewell Well Co					
121970064500	41.525896	-88.077534	1550	10	35	10	540	19130101	Joliet City (Van Buren St.)		Ohio Drlg Co		0	0	0	0
121970064800	41.536724	-88.058024	1589	11	35	10	558	19500101	E J & E Ry.		Miller, J. P. Art. Well		0	0	0	0
121970069300	41.517419	-88.052627	1303	14	35	10	580		Joliet Water Works		No Company		0	0	0	0
121970069400	41.522878	-88.056698	1608	14	35	10	555	19370101	Joliet City Of		Varner Well Drilling		0	0	0	0
121970069700	41.51849	-88.053893	1603	14	35	10	585	19410101	Prairie St Paper		Brandt & Heflin		0	0	0	0
121970070500	41.517678	-88.080493	1530	15	35	10	540		Joliet City Of (Spruce Slip)		Ohio Drlg Co		0	0	0	0
121970071600	41.522573	-88.085256	1560	16	35	10	540	19130101	Joliet City (Des Plaines St.)		Ohio Drlg Co		0	0	0	0
121970072000	41.52153	-88.091122	1565	16	35	10	540	19240101	Joliet City (Jasper St.)		Cater William		0	0	0	0
121970120300	41.48448	-88.125918	1495	30	35	10	543	19601001	Blockson Chem	6	Layne Western		0	0	0	0
121970124700	41.50383	-88.120181	1372	19	35	10	559	19210101	Amer Can Co	1	Geiger, S. B. & Son		0	0	0	0
121970124800	41.503955	-88.120549	1594	19	35	10	555	19420601	Amer Can Co	2	Miller, J. P. Art. Well		0	0	0	0
121970125000	41.507191	-88.11467	1586	20	35	10	565	19450101	Rockdale City of	2	Miller, J. P. Art. Well		0	0	0	0
121970125100	41.497301	-88.088999	1610	21	35	10	583		Amer Cyanamid	2	Layne Western		0	0	0	0
121970125200	41.499387	-88.089345	1604	21	35	10	572		Superior Alum Works	1	Miller, J. P. Art. Well					
121970125300	41.50583	-88.080743	1603	22	35	10	562	19290101	Superior Alum Works		Cater William					
121970125600	41.488077	-88.117994	1535	29	35	10	570	19530101	Blockson Chem	5	Layne Western Co., Inc.					
121970125700	41.49157	-88.11825	1509	29	35	10	530	19400101	Pub Serv Co	1	Geiger, S. B. & Son		0	0	0	0
121970125800	41.487193	-88.121243	1520	30	35	10	560		Blockson Chem	1	Miller, J. P. Art. Well					
121970125900	41.486779	-88.121002	1505	30	35	10	547	19410101	Blockson Chem	2	Geiger, S. B. & Son		0	0	0	0
121970126000	41.485087	-88.120838	1555	30	35	10	583	19510101	Blockson Chem	4	Layne Western		0	0	0	0
121970126100	41.488963	-88.136051	1540	30	35	10	535		Caterpillar Tract	1	Miller, J.P. Artesian Well Co.		0	0	0	0
121970126200	41.487464	-88.135396	1560	30	35	10	545		Caterpillar Tract	2	Miller, J.P. Artesian Well Co.		0	0	0	0
121970126400	41.553128	-88.002949	1660	5	35	11	645	19490101	Joliet City Of	1	Miller, J. P. Art. Well		0	0	0	0
121970126500	41.554484	-88.003999	1700	5	35	11	670		Joliet Site #5	2	Miller, J. P. Artesian Well Co.		0	0	0	0
121970126600	41.535518	-88.00299	1701	8	35	11	655	19500101	Joliet City Of	O-7	Miller, J. P. Art. Well		0	0	0	0
121970131300	41.516835	-88.054041	1639	14	35	10	595	19631001	Prairie St Paper	3	Wehling Well Works Inc.		0	0	0	0
121970203400	41.493438	-88.118058	1505	29	35	10	530	19710601	Commonwealth Edison	5	Wehling Well Works Inc.					
121970235500	41.493617	-88.12294	1525	30	35	10	510	19620101	Public Serv Co	3	Wehling Well Works, Inc.		0	0	0	0

TABLE 2.4-22C

ISGS PUBLIC GROUNDWATER SUPPLIES WITHIN 25 MILES (Continued)

	COORDINATES													PUMPING INFORMATION		
ISWS Well ID	LATITUDE	LONGITUDE	TOTAL DEPTH (FEET BGS[1])	SECTION	TOWNSHIP	RANGE	GROUND SURFACE ELEVATION (FEET AMSL[2])	COMPLETION DATE (YYYYMMDD)	WELL LOCATION/ OWNER	WELL NUMBER	INSTALLED BY	WELL TYPE	STATIC WATER LEVEL (FT BTOC[3])	PUMPING LEVEL (FT BTOC)	PUMPING RATE (GPM[4])	HOURS PUMPED
121973273300	41.525838	-88.125028	1670	7	35	10	0		Joliet City of		Miller J P Co					
121973568500	41.549405	-88.068037	1548	3	35	10	0	19980403	Joliet Correctional Center	4	Contract DeWatering Serv.,Inc.	COMM	710	775	725	24
121973842900	41.487235	-88.132199	1550	30	35	10	0	19970514	Caterpillar, Inc.	4	Layne-Western Co.		637	929	896	26
121973889900	41.496054	-88.138231	1525	19	35	10	530	20000306	Johns Manville Corporation	5	Buffington, G.	NPOT	624	680	600	0
121973950900	41.516835	-88.054041	1635	14	35	10	0		Prairie States Paper Mills	3	Wehling Well Works, Inc.		0	0	0	0
121973952500	41.496391	-88.122403	1525	19	35	10	0		Commonwealth Edison Co.	4	Wehling Well Works, Inc.		0	0	0	0
121973983500	41.546531	-88.073999	1580	3	35	10	0		Joliet Correctional Center	4		BUS	0	0	0	0
121973984900	41.505606	-88.115865	1575	20	35	10	0	19450101	Rockdale, Village of	1		MUNIC	0	0	0	0
121974164300	41.549299	-88.134693	1655	6	35	10	0	20050818	Joliet	23D	Layne-Western Co.	MUNIC	750	963	807	27
121974189200	41.548333	-88.062222	1663	2	35	10	0	20060331	Joliet, City of	24D	Brotcke Engineering	MUNIC	781	979	1360	24
120630128600	41.396947	-88.495809	1922	25	34	6	590		Hoge Abe		Hoge Abe					
120990150600	41.452848	-88.615464	1348	2	34	5	758	19600901	American Tel & Tel	1	Layne Western Co., Inc.	PRIV	320	360	10	2
120990156500	41.455928	-88.616578	1353	2	34	5	765	19610101	American Tel & Tel	4	Layne Western Co., Inc.	PRIV	320	360	10	2
120630090800	41.387769	-88.269681	1499	35	34	8	510	19570101	Dresden Nuc Pow Sta	1	Wehling Well Works Inc.					
120630134100	41.398263	-88.321828	1490	28	34	8	501	19620101	Desplaines Chem Co		Miller, J. P. Art. Well					
120630142900	41.455381	-88.258803	1508	1	34	8	611	19650501	City Of Minooka	3	Wehling Well Works Inc.					
120630146300	41.414021	-88.316296	1540	21	34	8	531	19670901	Amax Aluminum Co		Wehling Well Works Inc.					
120630152000	41.415059	-88.34037	1453	20	34	8	530	19681101	Northern Petro Co	1	Layne Western Co., Inc.					
120630201500	41.410622	-88.342306	1858	20	34	8	526	19691031		M-69-3	McCarthy Engr. & Const.					
120630203700	41.408057	-88.328204	1470	21	34	8	523	19701001	Northern Petro Co	2	Layne Western Co., Inc.					
120630205100	41.398546	-88.318585	1455	28	34	8	500	19701201	Northern Petro Co	5	Layne Western Co., Inc.					
120630205200	41.403426	-88.328151	1463	21	34	8	517	19700101	Northern Petro Co	3	Layne Western Co., Inc.					
120630210800	41.413055	-88.316284	1515	21	34	8	527	19711101	Amax Aluminum Co	2	Wehling Well Works Inc.					
120630218500	41.41144	-88.307107	1511	22	34	8	523	19721201	N Ill Gas Co	1	Layne Western Co., Inc.					
120630219600	41.411575	-88.301783	1519	22	34	8	530	19730401	N Ill Gas Co	2	Layne Western Co., Inc.					
120632306600	41.412982	-88.317494	1540	21	34	8	0	19890714	Alumax Mill Products	3	Wehling, Richard H.	COMM	335	0	0	0
120632411200	41.395053	-88.308601	1490	28	34	8	0		Des Plaines Chem. Co	2	Miller, J. P. Drlg Co.	PRIV	280	400	30	1
120632442000	41.434067	-88.269398	1601	12	34	8	605	20050609	Minooka, Village of	9	Water Well Solutions	MUNIC	427	612	1500	24
121970025100	41.443508	-88.160268	1407	11	34	9	540	19551001	Stephan Chemical	1	Layne Western		0	0	0	0
121970033100	41.447404	-88.172982	1420	11	34	9	571	19571001	Amoco Chem Corp	1	Layne Western		0	0	0	0
121970054200	41.448429	-88.17618	1405	10	34	9	573	19580501	Amoco Chemical	2	Layne Western		0	0	0	0
121970060300	41.392437	-88.145771	1627	25	34	9	606	19410101	Kankakee Ordn Works	8	Layne Western		0	0	0	0
121970060400	41.39932	-88.146048	1603	25	34	9	589	19410101	Kankakee Ordn Works	9	Layne Western		0	0	0	0
121970060500	41.406192	-88.146326	1571	25	34	9	591	19411001	Kankakee Ordn Works	10	Layne Western		0	0	0	0
121970060600	41.377243	-88.180683	1593	34	34	9	528	19410101	Kankakee Ordn Works	3	Layne Western		0	0	0	0
121970060700	41.377124	-88.187975	1554	34	34	9	522	19410101	Kankakee Ordn Works	4	Layne Western		0	0	0	0
121970060800	41.377367	-88.164341	1598	35	34	9	539	19410301	Kankakee Ordn Works	1	Layne Western		0	0	0	0
121970060900	41.377254	-88.173364	1605	35	34	9	532	19410101	Kankakee Ordn Works	2	Layne Western		0	0	0	0
121970061100	41.378701	-88.144999	1653	36	34	9	577	19410801	Kankakee Ordn Works	6	Layne Western		0	0	0	0
121970061200	41.385583	-88.145393	1655	36	34	9	601	19410101	Kankakee Ordn Works	7	Layne Western		0	0	0	0
121970061500	41.378422	-88.128613	1710	31	34	10	628	19420101	Kankakee Ordanance	12	Layne Western		0	0	0	0
121970121200	41.44623	-88.161567	1402	11	34	9	530	19601201	Stephan Chemical	2	Layne Western		0	0	0	0
121970132800	41.444811	-88.174458	1400	11	34	9	572	19640601	Amoco Chemicals	3	Milaeger Well Drlg.		0	0	0	0
121970181500	41.407579	-88.212374	1555	21	34	9	520	19700201	Durkee Glidden	1	Wehling Well Works Inc.		0	0	0	0
121970184500	41.412387	-88.188452	1578	22	34	9	556	19700401	Mobil Oil Co	1	Wehling Well Works, Inc.		0	0	0	0

TABLE 2.4-22C

ISGS PUBLIC GROUNDWATER SUPPLIES WITHIN 25 MILES (Continued)

	COORDINATES													PUMPING INFORMATION		
ISWS Well ID	LATITUDE	LONGITUDE	TOTAL DEPTH (FEET BGS^1)	SECTION	TOWNSHIP	RANGE	GROUND SURFACE ELEVATION (FEET AMSL^2)	COMPLETION DATE (YYYYMMDD)	WELL LOCATION/ OWNER	WELL NUMBER	INSTALLED BY	WELL TYPE	STATIC WATER LEVEL (FT BTOC^3)	PUMPING LEVEL (FT BTOC)	PUMPING RATE (GPM^4)	HOURS PUMPED
121970274000	41.383657	-88.188285	1593	34	34	9	530	19720801	Chicago-Joliet Livestock Market	2	Wehling Well Works, Inc.		0	0	0	0
121970337100	41.436185	-88.128155	1580	7	34	10	618	19721201	Peoples Gas	1	Layne Western		0	0	0	0
121972407800	41.438593	-88.131662	1597	7	34	10	615	19731101	Peoples Gas Light	1	Wehling Well Works, Inc.		0	0	0	0
121972408000	41.44303	-88.160164	1410	11	34	9	520	19730501	Stephan Chemical	3	Layne Western		0	0	0	0
121972415500	41.403716	-88.203386	1605	28	34	9	535	19740201	Dow Chem Usa	1	Wehling Well Works Inc.					
121972559100	41.45126	-88.177639	1415	3	34	9	570	19751201	Amoco Chemical	4	Wehling Well Works, Inc.		0	0	0	0
121972609600	41.406758	-88.212144	1555	21	34	9	0	19770301	Durkee Glidden	2	Wehling, R. H.					
121972623900	41.437195	-88.118806	1630	7	34	10	620	19771001	Carbonic Corp	1	Layne Western Co., Inc.		0	0	0	0
121973287700	41.417823	-88.196939	1573	21	34	9	545	19650901	Rexall Chem. Co.	1	Wehling Well Works, Inc.		191	588	950	0
121973676700	41.395537	-88.241901	1647	30	34	9	603	19930526	Channahon, Village of	4	Wehling, Richard H.	MUNIC	395	700	1000	24
121973725200	41.415288	-88.106438	1725	20	34	10	640	19961022	Elwood, Village of	6	Buffington, G.	MUNIC	510	784	501	24
121973728100	41.449314	-88.159436	1415	11	34	9	0	19960724	Stephan Chemical	4	Buffington, G.	INDUS	615	751	600	24
121973838300	41.392961	-88.130629	1665	30	34	10	0	19980604	U.S. Dept. Veterans Affrs	1	Buffington, G.	IRRIG	460	558	506	0
121973946200	41.406758	-88.212144	1560	21	34	9	0		Glidden Durkee	2	Wehling, Richard H.		0	0	0	0
121973994100	41.458764	-88.184041	1460	3	34	9	0	20000413	Joliet, City of	18	Layne-Western Co.	NCPUB	585	747	1000	26
121974066500	41.416073	-88.183257	1597	22	34	9	0	20021226	Exxon Mobil Refinery	2	Layne-Western Co.	NPOT	362	564	1241	0
121974157400	41.41612	-88.181765	1595	22	34	9	0	20050617	Exxon Mobil Oil Corp.	3	Buffington, G.	COMM	460	625	1209	0
121974203500	41.404002	-88.241851	1625	30	34	9	606	20060317	Channahon, Village of	6	Layne-Western Co.	MUNIC	425	597	1102	24
121974206600	41.445837	-88.180758	1490	10	34	9		20061103	Flint Hills Resources, LP	5	Water Well Solutions		577	955	450	3
121974240400	41.419777	-88.19351	1589	22	34	9	550	20070817	Exxon Mobil Oil Corp.	4	Layne-Western Co.	NPOT	465	783	1445	24
121973676800	41.431184	-87.993602	1703	17	34	11	0	19930331	Manhattan, Village of	6	Wehling, Richard H.	MUNIC	508	728	0	24
121973895000	41.421152	-87.973857	1770	21	34	11	700	20000828	Manhattan, Village of	7		MUNIC	497	595	1068	24
121974198500	41.440278	-87.982222	1725	9	34	11	690	20060701	Manhattan, Village of	8	Edward Hall - Web Well & Pump	MUNIC	540	898	1196	37
120630002800	41.305713	-88.564434	1510	29	33	6	606	19360401	Dupont Denemours		Gray, F. M., Jr., Inc.	MUNIC	635	690	890	24
120630084900	41.305871	-88.562797	1530	29	33	6	610	19550101	Dupont	6	Milaeger Well Drlg.	MUNIC	635	690	890	24
120630105300	41.305871	-88.562797	1545	29	33	6	602	19400101	Dupont Denemours	3	Milaeger Well Drlg.					
120632411700	41.310357	-88.555283	1545	29	33	6	606	19430527	Orica Nitrogen Survey	3		NCPUB	93	226	25	830
120990015300	41.308711	-88.598138	1447	25	33	5	510	19420101	Chi'Go Bridge & Iron	2	Geiger, S. B. & Son	PRIV	320	360	10	2
120990235700	41.339645	-88.70497	1466	7	33	5	655	19720101	City Of Marseilles	4	Miller, J. P. Art. Well					
120992506800	41.323664	-88.611695	1445	23	33	5	635	19930831	Seneca, Village of #3		Layne-Western Co.	MUNIC	208	280	250	8
120992642400	41.297221	-88.643959	1450	27	33	5	0	19000101	Marseilles	7		NCPUB	0	0	0	0
120992642500	41.304345	-88.656282	1450	28	33	5	0	19970101	Marseilles	6		NCPUB	0	0	0	0
120630107900	41.358215	-88.425045	1485	4	33	7	523	19150101	Morris City	3						
120630108000	41.361798	-88.429821	1462	4	33	7	505	19540101	Morris City	5	Milaeger Well Drlg.					
120630109500	41.35672	-88.427444	1501	9	33	7	510	19380101	Morris City	4	Mileage & Smyth					
120630222400	41.348741	-88.352708	1513	7	33	8	526	19730401	Commonwealth Edison	3	Wehling Well Works Inc.					
120630222500	41.350529	-88.352905	1477	7	33	8	525	19730301	Commonwealth Edison	2	Wehling Well Works Inc.					
120630222600	41.353539	-88.354513	1510	7	33	8	515	19730301	Commonwealth Edison	1	Wehling Well Works Inc.					
120630222700	41.350547	-88.362874	1495	7	33	8	505	19730501	Commonwealth Edison	4	Wehling Well Works Inc.					
120632236600	41.293501	-88.286723	1785	34	33	8	565	19780601	Coal City	5	Layne Western Co., Inc.					
120632335500	41.361685	-88.437293	1450	4	33	7	0	19931031	Morris, City of	6	Albrecht, S. Dean	TH	194	389	1600	15
120632429800	41.358226	-88.425053	1449	4	33	7	0	20020610	Morris, City of	7	Meadow Equipment	MUNIC	209	0	0	0

TABLE 2.4-22C

ISGS PUBLIC GROUNDWATER SUPPLIES WITHIN 25 MILES (Continued)

	COORDINATES		TOTAL DEPTH				GROUND SURFACE ELEVATION	COMPLETION DATE				WELL	STATIC WATER LEVEL	PUMPING INFORMATION		
ISWS Well ID	LATITUDE	LONGITUDE	(FEET BGS[1])	SECTION	TOWNSHIP	RANGE	(FEET AMSL[2])	(YYYYMMDD)	WELL LOCATION/ OWNER	WELL NUMBER	INSTALLED BY	TYPE	(FT BTOC[3])	PUMPING LEVEL (FT BTOC)	PUMPING RATE (GPM[4])	HOURS PUMPED
120632444600	41.355691	-88.458835	1375	8	33	7	0	20050711	Exelon Midwest Fire Trainin	2	Kerry, Charles M.	NCPUB	267	440	240	24
121970059100	41.360813	-88.076862	1672	9	33	10	647		Elwood Ordnance		Miller, J. P. Art. Well		0	0	0	0
121970059200	41.364557	-88.084262	1645	9	33	10	640	19410101	Elwood Ordnance	2	Miller, J. P. Art. Well		0	0	0	0
121970068000	41.36084	-88.135488	1635	12	33	9	575	19420101	Kankakee Ordnance Works	11	Layne Western		0	0	0	0
121970071500	41.308272	-88.146212	1565	25	33	9	540	19360101	Wilmington City	2	Varner C W		0	0	0	0
121970133100	41.304774	-88.1471	1575	36	33	9	530	19641101	Wilmington City of	3	Wehling Well Works, Inc.		0	0	0	0
120990234900	41.239946	-88.666284	1620	17	32	5	711	19720501	Commonwealth Edison		Wehling Well Works Inc.					
120992245100	41.247349	-88.669197	1629	17	32	5	711	19740101	Commonwealth Edison	1	Wehling Well Works Inc.					
121970001000	41.263928	-88.218262	1647	8	32	9	577	19370101	Braidwood City	1	Varner C W					
121972484600	41.244594	-88.230882	1760	19	32	9	597	19741001	Commonwealth Edison	1	Wehling Well Works, Inc.		0	0	0	0
121972722600	41.2803	-88.220957	1732	5	32	9	0	19790731	Braidwood, City of	3	Wehling, Richard H.	MUNIC	237	415	1250	8
121973118900	41.222538	-88.187879	1690	28	32	9	0	19811029	Commonwealth Edison Co.		Wehling, Wendell E.	COMM	0	0	0	0
121973708700	41.212446	-88.025161	1700	36	32	10	603	19860826	Capital Development Board		Wehling, Wendell E.	NCPUB	0	0	0	0
121974267700	41.244624	-88.227214	1750	19	32	9		20090422	Exelon-Braidwood		Wehling, Richard W.	NPOT	288	335	610	24
120910040500	41.253172	-87.835061	1292	15	32	12	0	19130101	Manteno City Well		Ohio Drlg Co					
120910040600	41.233895	-87.812017	1760	26	32	12	675	19300101	Manteno St Hosp	1	Miller, J. P. Art. Well					
120630151600	41.186958	-88.301279	1933	4	31	8	587	19681001	Vlge Of Gardner	4	Layne Western Co., Inc.					
120632333600	41.187676	-88.305034	1929	4	31	8	0	19930618	Village of Gardner #5		Wehling, Richard H.	MUNIC	219	281	0	8
120632411300	41.172875	-88.275535	1220	11	31	8	0	20010109	South Wilmington	5	Richard W. Wehling	NCPUB	232	417	0	8
121050003600	41.094583	-88.4715	1201	1	30	6	648	19480601	Womens Reform	2	Milaeger Well Drlg.	COMM	280	300	1	2
121050047900	41.09456	-88.473903	1203	1	30	6	648	19300101	Womens Reform	1	Cater William	COMM	280	300	1	2
121050048300	41.085657	-88.438117	2110	9	30	7	640	19220101	Chgo & Alton R R		No Company	COMM	280	300	1	2
120910009900	41.042171	-88.120087	2038	31	30	10	661	19560801	Armstrong	2	Natural Gas Storage Company of IL	IRRIG	50	400	200	3
121050035400	40.998772	-88.518881	1298	10	29	6	692		Odell City		Jverndt W	COMM	280	300	1	2
121050041300	41.000605	-88.516408	1935	10	29	6	700		Odell City	3	Layne Western Co., Inc.	COMM	280	300	1	2
121050055800	40.998772	-88.518881	1584	10	29	6	0	19120101	Odell Village		Miller, J. P. Art. Well	COMM	280	300	1	2
121052120300	40.956366	-88.566389	1330	29	29	6	0	19650101	Scully Peter				0	0	0	0
121052252200	40.997173	-88.521729	1935	10	29	6	0	19901106	Odell, Village of #4		Wehling, Richard H.	MUNIC	235	237	0	5
121050038600	40.878692	-88.270298	1670	23	28	8	715		Cullom City Well		No Company	COMM	280	300	1	2

Note: [1] BGS - below ground surface
[2] AMSL - above mean sea level
[3] BTOC - below top of casing
[4] GPM - gallons per minute

All townships and ranges within the 25-mile radius were included within the search.

Therefore, some wells listed may be outside of the 25-mile radius.

TABLE 2.4-22D

ISWS PUBLIC GROUNDWATER SUPPLIES WITHIN 25 MILES

ISWS WELL ID	COORDINATES		TOTAL DEPTH (FEET BGS[1])	SECTION	TOWNSHIP	RANGE	PLOT	GROUND SURFACE ELEVATION (FEET AMSL[2])	COMPLETION DATE (YYYYMMDD)	WELL LOCATION/OWNER	INSTALLED BY	WELL USE
	LATITUDE	LONGITUDE										
52630			1250	24	29N	09E	1A		19600000	HUGES	WEHLING	DO
401679			1433	29	33N	06E	3D		19280000	ORICA NITROGEN	J. OTIS HEFLIN	IC
401680			1545	29	33N	06E	4E	606	19400400	ORICA NITROGEN	R. E. MILAEGER WELL DRILLING CO.	IC
401683			1530	29	33N	06E	4E	610	19550000	ORICA NITROGEN	R. E. MILAEGER WELL DRILLING CO.	IC
401677			1515	29	33N	06E	5E		19280000	ORICA NITROGEN	J. O. HEFFLIN	IC
411965			1554	01	35N	08E	5H		20060323	JOLIET	LAYNE-WESTERN	CS
411893			1533	11	35N	08E	4G		2005	JOLIET	LAYNE WESTERN	CS
411964			1523	12	35N	08E	7H		2006	JOLIET	LAYNE-WESTERN	CS
411496			1520	26	35N	08E	2D	616	2005	MINOOKA	WATER WELL SOLUTIONS DRILLING	CS
421452			1427	06	36N	07E				YORKVILLE		CS
411219			1527	10	36N	07E	1G		2004	YORKVILLE	LAYNE-WESTERN	CS
411342			1555	25	36N	08E	8A	665	2003	JOLIET	LAYNE-WESTERN	CS
411394			1556	35	36N	08E	1A	660	2003	JOLIET	LAYNE-WESTERN	CS
74788			2196	03	30N	09E	8F		19650000	NAT GAS PIPELINE CO		IC
74787			1672	03	30N	09E	8G		19650000	NAT GAS PIPELINE CO		IC
55907			1922	25	34N	06E	7G		18750000	HOGE A	ANDERSON	DO
406629			1290	15	32N	12E	7B	685	1913	MANTENO	OHIO DRILLING CO	CS
406635			1760	26	32N	12E	5G	677	1930	ILL VETERAN'S HOME-MANTENO		
402327			1620	17	32N	05E	1A	714	1972	EXELON - LASALLE CO STATION	WEHLING WELL WORKS	
402326			1629	17	32N	05E	2F	704	1974	EXELON - LASALLE CO STATION	WEHLING WELL WORKS	
81063			1620	17	32N	05E			19720525	CON ED CO		IC
407057	41.3394444	88.7011111	1466	07	33N	05E	6A	688	1972	MARSEILLES	J P MILLER ART WELL	
410328	41.3237500	88.6094444	1445	23	33N	05E	1G	635	1993	SENECA	ALBRECHT WELL DRLG	
411313			1330	28	33N	05E	4E	504	1998	MARSEILLES SOUTH	MEADOW EQUIPMENT	
402394			1348	02	34N	05E	2H			AMERICAN TELEPHONE & TELEGRAPH		
402395			1353	02	34N	05E	3H			AMERICAN TELEPHONE & TELEGRAPH		
382117			2180	32	34N	05E			19220000	R.N. PEDDICORD		DO
407203			1670	23	28N	08E	4A	685	1914	CULLOM	--	
278311			1360	13	28N	08E	4H		19610000	MRS. G. LONGBOTTOM/GLEN EHLERS	BOLLIGER	DO
410074	40.9997222	88.5219444	1935	10	29N	06E	6D	703	1990	ODELL	WEHLING WELL WORKS	
407212			1298	10	29N	06E	7F	720	1898	ODELL	OHIO DRILLING CO	
407213	41.0000278	88.5269444	1935	10	29N	06E	8E	710	1951	ODELL	LAYNE-WESTERN CO	
278604			1330	29	29N	06E	8E		19650000	PETER SCULLY	FRENCH	DO
278601			1330	30	29N	06E	1E		19650401	PETER SCULLY	FRENCH	DO
407216	41.0951389	88.4669444	1201	01	30N	06E	1A	648	1948	DWIGHT CORRECTIONAL CENTER	MILAEGER WELL & PUMP	
407217	41.0950000	88.4733333	1203	01	30N	06E	2A	645	1930	DWIGHT CORRECTIONAL CENTER	WILLIAM H CATER	
189059			2110	09	30N	07E	8C		19200000	CHICAGO & ALTON RAILROAD	J.P. MILLER	IC
412019			1730	09	34N	11E	7A	690	2006	MANHATTAN	LAYNE-WESTERN	CS
410378	41.4472222	87.9975000	1703	17	34N	11E	5D	685	1993	MANHATTAN	L-W CO-WEHLING DIV	CS
400177			1770	21	34N	11E	5F		2000	MANHATTAN	LAYNE-WESTERN	CS
375134			2000	23	34N	11E	7E		20050907	PHILLIP STEVENS	COMAR DRILLING/DAVE STINNETT	DO

TABLE 2.4-22D

ISWS PUBLIC GROUNDWATER SUPPLIES WITHIN 25 MILES (Continued)

ISWS WELL ID	COORDINATES LATITUDE	COORDINATES LONGITUDE	TOTAL DEPTH (FEET BGS[1])	SECTION	TOWNSHIP	RANGE	PLOT	GROUND SURFACE ELEVATION (FEET AMSL[2])	COMPLETION DATE (YYYYMMDD)	WELL LOCATION/OWNER	INSTALLED BY	WELL USE
151656			1605	28	34N	12E	5G		19740208	DOW CHEMICAL	WEHLING	IC
309464			1850	36	34N	12E	8H		19990119	WAYNE AANERUD	FRANK SHARPE	DO
409239	41.5451389	88.1455556	1623	01	35N	09E	3E	619	1975	JOLIET	WEHLING WELL WORKS	CS
411671			1520	03	35N	09E	5G		1999	JOLIET	LAYNE-WESTERN	CS
411494			1458	10	35N	09E	3A		1963	HOLIDAY INN - JOLIET	J.P. MILLER ARTESIAN	
411495			1556	10	35N	09E	3A	570	1966	HOLIDAY INN - JOLIET		
409245	41.5250000	88.1766667	1440	10	35N	09E	8A	600	1976	IMPERIAL MHP	K & K WELL DRILLING	CS
409247	41.5241667	88.1561111	1572	11	35N	09E	1B	610	1970	JOLIET	J P MILLER ART WELL	CS
411611			1618	12	35N	09E	1A		2005	JOLIET	LAYNE-WESTERN	CS
421920			1631	19	35N	09E	5A		20071116	SHOREWOOD	WATER WELL SOLUTIONS	CS
432145			1631	29	35N	09E	4H		20080130	SHOREWOOD	WATER WELL SOLUTIONS	CS
409251	41.4769444	88.2052778	1429	33	35N	09E	4H	575	1966	UTILITIES INC - CAMELOT UTILITIES INC	SHAVER DRL/K & K DRL	
152929			1520	03	35N	09E	2A		19571202	DUPAGE RIVER FARM	MILLER	IC
253432			1455	10	35N	09E			19900203	ALAN DWORKIN #3	FYKES	DO
153069			1460	15	35N	09E	2H		19660000	JOHNSON MOTEL	MILLER	IC
152833			1445	20	35N	09E	4E		19751128	LEACH J	LOCKPORT	DO
404056			1556	25	35N	09E	1E	547	1960	CENTERPOINT PROPERTIES	J P MILLER ART WELL	NR
412076			1663	02	35N	10E	8F		2006	JOLIET	WATER WELL SOLUTIONS	CS
405567	41.5471389	88.0808333	1548	03	35N	10E	2F	550		JOLIET CORRECTIONAL CENTER		CS
409255	41.5467778	88.0764167	1600	03	35N	10E	4E	560	1948	JOLIET CORRECTIONAL CENTER	J P MILLER ART WELL	CS
404075			1595	04	35N	10E	2H			GERDAU AMERISTEEL - JOLIET MILL		IC
411670			1655	06	35N	10E	6G	610	2005	JOLIET	LAYNE-WESTERN	CS
409258	41.5094444	88.1269444	1671	07	35N	10E	4B	647	19640900	JOLIET	J P MILLER ART WELL	CS
404078			1639	14	35N	10E	5D	595	1963	IVEX JOLIET		IC
404077			1603	14	35N	10E	5E	581		IVEX JOLIET		IC
409291	41.5245833	88.0566667	1608	14	35N	10E	6H	564	1937	JOLIET	VARNER WELL DRILLING	CS
404692			1372	19	35N	10E	1F	555	1921	NATIONAL BOTTLE MANUFACTURING	S B GEIGER & CO/-	
404693			1594	19	35N	10E	1F	555	1942	NATIONAL BOTTLE MANUFACTURING	J P MILLER ART WELL	
404188			1525	19	35N	10E	2B			MIDWEST GENERATION - JOLIET ST 29 UNITS 7-8		IC
410982			1525	19	35N	10E	8B	530	2000	JOHNS MANVILLE INTERNATIONAL I	LAYNE-WESTERN	IC
404182			1487	20	35N	10E	6A			MIDWEST GENERATION - JOLIET ST 9 UNITS 1-6		IC
409302	41.5072222	88.1133333	1586	20	35N	10E	7G	556	1945	ROCKDALE	J P MILLER ART WELL	CS
404046			1604	21	35N	10E	4C	580	1920	C & S CHEMICAL	J P MILLER ART WELL	
404647			1535	29	35N	10E	8E	565	1952	KEATING RESOURCES	LAYNE-WESTERN CO	IC
404183			1505	29	35N	10E	8H	527	19710601	MIDWEST GENERATION - JOLIET ST 9 UNITS 1-6	WEHLING	IC
404072			1555	30	35N	10E	1C	580	1950	KEATING RESOURCES	LAYNE-WESTERN CO	IC
404187			1525	30	35N	10E	2H			MIDWEST GENERATION - JOLIET ST 29 UNITS 7-8		IC
402386			1498	30	35N	10E	3C	550	1960	OLIN CORP JOLIET PLANT	LAYNE-WESTERN CO	IC

TABLE 2.4-22D

ISWS PUBLIC GROUNDWATER SUPPLIES WITHIN 25 MILES (Continued)

ISWS WELL ID	COORDINATES		TOTAL DEPTH	SECTION	TOWNSHIP	RANGE	PLOT	GROUND SURFACE ELEVATION	COMPLETION DATE	WELL LOCATION/OWNER	INSTALLED BY	WELL USE
	LATITUDE	LONGITUDE	(FEET BGS[1])					(FEET AMSL[2])	(YYYYMMDD)			
404055			1420	30	35N	10E	6E	546	1950	CATERPILLAR INC - JOLIET	J P MILLER ART WELL	IC
405460			1550	30	35N	10E	6E	542	1997	CATERPILLAR INC - JOLIET	LAYNE-WESTERN CO	IC
154805			1215	03	35N	10E			19410000	AM STEELE CO		IC
221192			1600	03	35N	10E			19180000	ILLINOIS STATE PENITENTIARY		ST
154800			1500	03	35N	10E	4C		19020000	JA HEGGIE(UNIV HEAD CO)		IC
154806			1602	03	35N	10E	6B		19130000	AM STEELE AND WIRE CO		IC
160815			1600	04	35N	10E			19260000	JOLIET TERMINALS	CATER	IC
154826			1559	04	35N	10E			19450000	JOLIET INDUSTRIES	MILLER	IC
154824			1596	04	35N	10E	2G		19260000	JOLIET TERMINALS(IND)	CATER	IC
160843			1600	10	35N	10E			19420000	ACME BREWING CO		IC
299018			1575	10	35N	10E			19020000	FRED SEHING BREWING CO.	J.P.MILLER	IC
154785			1507	10	35N	10E			19350000	LINDBERG CO	HEFFIN	IC
154784			1530	10	35N	10E	1A		19450400	PRATT MGF CO	MILLER	IC
154780			1460	10	35N	10E	4C		19380000	MEDOOW GOLD CREAMERY	SEWELL CO	IC
154779			1350	10	35N	10E	5B		19340000	BOHEMIAN BREWING CO	GREY	IC
153845			1587	11	35N	10E			19491200	EJ AND ERY	MILLER	DO
264081			1589	11	35N	10E			19500000	S.J. & E. RY	MILLER ARTESIAN WELL	DO
299019			1303	14	35N	10E			00000000	JOLIET WATERWORKS		CS
220939			1921	19	35N	10E	1G		19430900	AMERICAN CAN CO #3		IC
154818			1495	20	35N	10E	6A		19580200	COMMONWEALTH EDISON	MILEAGER	IC
154774			1603	22	35N	10E	7G		19290000	AMER INST OF LAUNDERING	CATIO	IC
154815			1509	29	35N	10E	8H		19400000	COMMONWEALTH EDISON	GEIGER	IC
154817			1508	29	35N	10E	8H		19410000	COMMONWEALTH EDISON		IC
154816			1558	33	35N	10E			19310000	COMMONWEALTH EDISON	MILLER	IC
154765			1588	35	35N	10E			19580000	FAIRMONT SCHOOL	WEHLING	SC
409311	41.5411111	88.0019444	1660	05	35N	11E	7H	648	1949	JOLIET	J P MILLER ART WELL	CS
409315	41.5338889	88.0022222	1700	08	35N	11E	8H	674	1950	JOLIET	J P MILLER ART WELL	CS
409349	41.6182500	88.1989167	1480	10	36N	09E	7D	612	1956	PLAINFIELD	LAYNE-WESTERN CO	CS
412272			1500	11	36N	09E	7A		20050218	OZINGA BROS INC - MOKENA PLANT	LAYNE-WESTERN	IC
410205	41.5944444	88.2047222	1508	16	36N	09E	2A	604	1991	PLAINFIELD	WEHLING WELL WORKS	CS
409355	41.5733333	88.1673611	1557	25	36N	09E	6D	602	1975	JOLIET	WEHLING WELL WORKS	CS
411395			1580	31	36N	09E	5A			JOLIET		CS
400181			1525	31	36N	09E	6A	633	2000	JOLIET	LAYNE-WESTERN	CS
405554			1566	32	36N	09E	4D		19970912	JOLIET	LAYNE WESTERN CO	CS
221148			1380		36N	09E			19170000	PLAINFIELD OLD #2		CS
218154			1560	06	36N	09E	3B		19880815	ROUSONELOUS FARMS	WEHLING	IR
157636			1958	23	36N	09E	1H		19440000	HERREN OIL CO		IC
404190			1500	02	36N	10E	7F		19740322	MIDWEST GENERATION - ROMEOVILLE	WEHLING WELL WORKS	IC
404192			1507	02	36N	10E	8F			MIDWEST GENERATION - ROMEOVILLE		IC
409363	41.6390000	88.0978056	1524	04	36N	10E	6G	670	1964	ROMEOVILLE	LAYNE-WESTERN CO	
405297	43.6344444	88.1365278	1555	06	36N	10E	6F	650	1996	ROMEOVILLE	LAYNE-WESTERN CO	

TABLE 2.4-22D

ISWS PUBLIC GROUNDWATER SUPPLIES WITHIN 25 MILES (Continued)

ISWS WELL ID	COORDINATES		TOTAL DEPTH (FEET BGS^1)	SECTION	TOWNSHIP	RANGE	PLOT	GROUND SURFACE ELEVATION (FEET AMSL^2)	COMPLETION DATE (YYYYMMDD)	WELL LOCATION/OWNER	INSTALLED BY	WELL USE
	LATITUDE	LONGITUDE										
410968			1540	07	36N	10E	6A		1998	ROMEOVILLE	LAYNE-WESTERN	
411268			1475	21	36N	10E	5D		20001128	PRAIRIE BLUFF GOLF COURSE	BOETSCH	IR
404083			1546	28	36N	10E	1B	563	1980	ALCAN POWDER / CHEMICAL - TOYO	WEHLING WELL WORKS	
158077			1441		36N	10E			19580000	NORTH ILL GAS CO		IC
158087			1505	02	36N	10E			19570000	COMMONWEALTH EDISON	NEELY	IC
158076			1507	02	36N	10E	8F		19570300	PUBLIC SERV CO	NEELY	IC
158075			1536	02	36N	10E	8H		19520000	PUBLIC SERV CO	NEELY	IC
158074			1537	03	36N	10E	7E		19520500	PUBLIC SERV CO	NEELY	IC
220607			1600	23	36N	10E			19200000	LOCKPORT		CS
220613			1475	23	36N	10E	6D		19410100	LOCKPORT #2		CS
220605			1650	23	36N	10E	8D		19150000	LOCKPORT #1		CS
220611			1365	26	36N	10E			00000000	DELLWOOD PARK WELL		PK
158080			1502	28	36N	10E			19290000	WESTERN UNITED GAS & ELECTRIC		IC
158081			1500	28	36N	10E			19290000	WESTERN UNITED GAS & ELECTRIC		IC
158071			1505	28	36N	10E	1B		19190000	ALCAN POWDER/ CHEMICAL- TOY0		IC
158073			1558	33	36N	10E	6H		19320000	PUBLIC SERV NORTH ILL	MILLER	IC
158386			1558	33	36N	10E	6H		19320000	COMMONWEALTH EDISON	MILLER	IC
158393			2076	34	36N	10E			19410000	STEEL MILL	WALLON	IC
158068			1600	34	36N	10E			19410000	STEELE CO COKE PLANT		IC
220612			2069	34	36N	10E			00000000	CARNEGIE ILL STEEL CO #2		IC
160973			2069	34	36N	10E			19580000	CARNEGIE STEEL CO		IC
158069			1605	34	36N	10E	5C		19430000	STEELE CO COKE PLANT(DEEPENED8	MILLER	IC
158078			1588	35	36N	10E	8G		19580912	FAIRMONT SC DIST	WEHLING	SC
405925	41.1891667	88.3077778	1933	04	31N	08E	1A	588	1968	GARDNER	LAYNE-WESTERN CO	
410240	41.1877778	88.3055556	1929	04	31N	08E	2A	587	1993	GARDNER	L-W CO-WEHLING DIV	
400036	41.1728750	88.2755350	1220	11	31N	08E	6A		20010109	SOUTH WILMINGTON	RICHARD W. WEHLING	CS
405968	41.3577778	88.4258333	1485	04	33N	07E	2A	522	1915	MORRIS	LAYNE-WESTERN	CS
405969	41.3625000	88.4297222	1462	04	33N	07E	4C	506	1954	MORRIS	MILAEGER WELL & PUMP	CS
410360	41.3622222	88.4361111	1451	04	33N	07E	6C	525	1993	MORRIS	ALBRECHT WELL DRLG	CS
412026			1375	08	33N	07E	8H		2005	EXELON MIDWEST FIRE TRAINING ACADEMY	MEADOW EQUIPMENT	IC
410959	41.3569444	88.4305556	1449	09	33N	07E	4H	510	2002	MORRIS	MEADOW EQUIPMENT	CS
405976	41.2936111	88.2869444	1785	34	33N	08E	1D	560	1978	COAL CITY	LAYNE-WESTERN CO	
405974			1788	34	33N	08E	6F	560	1893	CARBON HILL	--	
405989	41.4561111	88.2525000	1508	01	34N	08E	3E	610	1965	MINOOKA	WEHLING WELL WORKS	CS
411799			1601	12	34N	08E	8A	605	20050609	MINOOKA	WATER WELL SOLUTIONS DRILLING DIVISION	CS
401644			1453	20	34N	08E	2E		1968	LYONDELL/EQUISTAR - MORRIS	LAYNE-WESTERN	IC
429358			1470	20	34N	08E	3C			AKZO NOBEL SURFACE CHEMISTRY		IC
401623			1515	21	34N	08E	3F	528	1971	SAPA INDUSTRIAL EXTRUSIONS	E.C. WEHLING	IC
401621			1540	21	34N	08E	3G	530	1967	SAPA INDUSTRIAL EXTRUSIONS		IC
401624			1540	21	34N	08E	4F	528	1989	SAPA INDUSTRIAL EXTRUSIONS		IC

TABLE 2.4-22D

ISWS PUBLIC GROUNDWATER SUPPLIES WITHIN 25 MILES (Continued)

ISWS WELL ID	COORDINATES		TOTAL DEPTH (FEET BGS[1])	SECTION	TOWNSHIP	RANGE	PLOT	GROUND SURFACE ELEVATION (FEET AMSL[2])	COMPLETION DATE (YYYYMMDD)	WELL LOCATION/OWNER	INSTALLED BY	WELL USE
	LATITUDE	LONGITUDE										
401647			1463	21	34N	08E	8A		1971	LYONDELL/EQUISTAR - MORRIS	LAYNE-WESTERN	IC
401646			1470	21	34N	08E	8C		1970	LYONDELL/EQUISTAR - MORRIS	LAYNE-WESTERN	IC
401654			1519	22	34N	08E	6E		19720000	NORTH IL GAS SNG PLANT		IC
401653			1511	22	34N	08E	8E		19720000	NORTH IL GAS SNG PLANT		IC
401649			1492	28	34N	08E	1D	490		LYONDELL/EQUISTAR - MORRIS		IC
401648			1455	28	34N	08E	5F		1970	LYONDELL/EQUISTAR - MORRIS	LAYNE-WESTERN	IC
401674			1500	35	34N	08E	1E	510	1957	EXELON - DRESDEN STATION		IC
401673			1499	35	34N	08E	1G	515	1957	EXELON - DRESDEN STATION		IC
56459			1906	20	34N	08E			19780000	U P G INC	RECKTOR AND STONE	IC
56460			1925	20	34N	08E			19780000	U P G INC	RECTOR AND STONE	IC
56461			1925	20	34N	08E			19780000	U P G INC	RECKOR AND STONE	IC
56472			1492	28	34N	08E	1D		19620509	DES PLAINES CHEMICAL CO	ARTESIAN WELL CO(MIL	IC
56514			1500	35	34N	08E	5A		19570000	DRESDEN NUC POWER STAT	WHELING	IC
72629			1260	28	30N	10E	8D		00000000	TEXAS ILLINOIS	EARLE & HOLDER	IC
72630			1883	28	30N	10E	8D		19510400	TEXAS ILLINOIS	EARLE & HOLDER	IC
72633			1900	31	30N	10E	7G		00000000	NATURAL GAS STORAGE WELL		IC
72640			1404	32	30N	10E	2H		19510500	TEXAS ILLINEOS NATRUAL GAS	EAKLE & HOLDER	IC
72641			1861	32	30N	10E	2H		19510000	NATURAL GAS STORAGE CO. OF ILL	EAKLE & HOLDER	IC
72642			1710	32	30N	10E	2H		19510616	NATURAL GAS STORAGE CO. OF ILL	EAKLE & HOLDER	IC
74788			2196	03	30N	09E	8F		19650000	NAT GAS PIPELINE CO		IC
74787			1672	03	30N	09E	8G		19650000	NAT GAS PIPELINE CO		IC
409163	41.2805556	88.2213889	1733	05	32N	09E	6D	560	1979	BRAIDWOOD	WEHLING WELL WORKS	
409164	41.2641667	88.2191667	1647	08	32N	09E	5C	575	1937	BRAIDWOOD	C VARNER/W L THORNE	
409168			1700	36	32N	10E	2D	610	1986	KANKAKEE CORRECTIONAL CENTER	WEHLING WELL WORKS	ST
160754			1700	36	32N	10E	2D		19680800	ILL YTH CENTER	WEHLING	PK
409784			1652	01	33N	09E	5D	560	1942	MIDEWIN NATIONAL TALLGRASS PRAIRIE - USDA FS	VARNER WELL & PUMP	IC
409788			1644	12	33N	09E	1G	575	1942	MIDEWIN NATIONAL TALLGRASS PRAIRIE - USDA FS	LAYNE-WESTERN CO	IC
409173	41.3041667	88.1472222	1578	36	33N	09E	7H	530	1964	WILMINGTON	WEHLING WELL WORKS	CS
149179			1653	01	33N	09E			19420114	KANKEE ORDANCE WORKS	VARNER	IC
149217			1630	12	33N	09E	1G		19440000	US RUBBER CO 11		IC
409811	41.3593611	88.0771111	1672	09	33N	10E	1F	647	1941	MIDEWIN NATIONAL TALLGRASS PRAIRIE - USDA FS	J P MILLER ART WELL	IC
409783	41.3631944	88.0843611	1645	09	33N	10E	4H	640	1941	MIDEWIN NATIONAL TALLGRASS PRAIRIE - USDA FS	J P MILLER ART WELL	IC
149453			1672	09	33N	10E			19410000	ELWOOD ORDANCE PLANT		IC
149454			1645	09	33N	10E			19410000	ELWOOD ORD PLANT		IC
404053			1415	03	34N	09E	1A	570	19751231	FLINT HILLS RESOURCES	MILAEGER WELL & PUMP CO.	IC
400038			1460	03	34N	09E	4F		20000413	JOLIET	LAYNE-WESTERN	CS
404050			1405	10	34N	09E	1H	573	19580500	FLINT HILLS RESOURCES	LAYNE WESTERN CO.	IC
423302	41.4452444	88.1844708	1515	10	34N	09E	4F	576	20061103	FLINT HILLS RESOURCES	WATER WELL SOLUTIONS	IC

TABLE 2.4-22D

ISWS PUBLIC GROUNDWATER SUPPLIES WITHIN 25 MILES (Continued)

ISWS WELL ID	COORDINATES		TOTAL DEPTH	SECTION	TOWNSHIP	RANGE	PLOT	GROUND SURFACE ELEVATION	COMPLETION DATE	WELL LOCATION/OWNER	INSTALLED BY	WELL USE
	LATITUDE	LONGITUDE	(FEET BGS^1)					(FEET AMSL^2)	(YYYYMMDD)			
405373			1415	11	34N	09E	2A	535	1996	STEPAN CHEMICAL COMPANY	LAYNE-WESTERN CO	IC
404032			1410	11	34N	09E	2E	525	1973	STEPAN CHEMICAL COMPANY	LAYNE-WESTERN CO	IC
404031			1402	11	34N	09E	3D	520	19601209	STEPAN CHEMICAL COMPANY	LAYNE-WESTERN CO	IC
404049			1422	11	34N	09E	7G	571	19571022	FLINT HILLS RESOURCES	LAYNE-WESTERN COMPANY	IC
404052	41.4448110	88.1744580	1369	11	34N	09E	8F	570	19640601	FLINT HILLS RESOURCES	MILAEGER WELL & PUMP CO.	IC
404080			1573	21	34N	09E	2D		19650823	INEOS NOVA LLC	WEHLING WELL WORKS, INC.	IC
									2.4-76			
411911			1595	22	34N	09E	4F	562	2005	EXXON MOBIL OIL CORP - JOLIET REFINERY	LAYNE-WESTERN (AURORA)	IC
411133			1597	22	34N	09E	4F	545	2002	EXXON MOBIL OIL CORP - JOLIET REFINERY	LAYNE-WESTERN	IC
432165	41.3953333	89.2461167	1589	22	34N	09E	6F	550	20071024	EXXON MOBIL OIL CORP - JOLIET REFINERY	LAYNE-WESTERN	IC
404067			1578	22	34N	09E	7D	556	1970	EXXON MOBIL OIL CORP - JOLIET REFINERY	WEHLING	IC
409821			1627	25	34N	09E	5A	606	1941	ELWOOD	LAYNE-WESTERN CO	CS
409790			1603	25	34N	09E	5D	590	1941	ELWOOD	LAYNE-WESTERN CO	CS
409789			1571	25	34N	09E	5H	591	1941	ELWOOD	LAYNE-WESTERN CO	CS
404008			1605	28	34N	09E	5H	534	1974	DOW CHEMICAL	WEHLING WELL WORKS	
411969	41.4059167	88.2419167	1625	30	34N	09E	4H	606	20060317	CHANNAHON	LAYNE-WESTERN	CS
410382	41.3863889	88.2290278	1647	30	34N	09E	5D	603	1993	CHANNAHON	L-W CO-WEHLING DIV	CS
409814			1593	34	34N	09E	3A	528	1941	MIDEWIN NATIONAL TALLGRASS PRAIRIE - USDA FS	LAYNE-WESTERN CO	IC
409815			1551	34	34N	09E	7A	522	1941	MIDEWIN NATIONAL TALLGRASS PRAIRIE - USDA FS	LAYNE-WESTERN CO	IC
404027			1593	34	34N	09E	7D	530	1972	HAGER'S COUNTRY MARKET, INC	WEHLING WELL WORKS	
409812			1597	35	34N	09E	5A	539	1941	MIDEWIN NATIONAL TALLGRASS PRAIRIE - USDA FS	LAYNE-WESTERN CO	IC
409813			1612	35	34N	09E	8A	532	1941	MIDEWIN NATIONAL TALLGRASS PRAIRIE - USDA FS	LAYNE-WESTERN CO	IC
409817			1653	36	34N	09E	5A	578	1941	ELWOOD	LAYNE-WESTERN CO	CS
409818	42.3916667	88.1416667	1655	36	34N	09E	5E	601	1941	ELWOOD	LAYNE-WESTERN CO	CS
150362			1415	03	34N	09E	1A		19751231	AMACO CHEM CO	WHELING	IC
150302			1375	11	34N	09E	8H		19571100	AMOCO CHEMICAL CO		IC
150310			1407	14	34N	09E			19551000	STEPAN CHEM CO	LAYNE WESTERN	IC
150785			1555	21	34N	09E	8B		19770300	LODERS CROKLAAN	WEHLING	IC
294024			1647	30	34N	09E	5D		19930700	VILLAGE OF CHANNAHON #4	LAYNE-WESTERN	CS
150808			1648	36	34N	09E	5A		19420000	US RUBBER UNIROYAL CO	JP MILLER	IC
444377			1600	07	34N	10E				JOLIET		CS
411507			1581	07	34N	10E	5A			ELWOOD ENERGY LLC		IC
405294	41.4155556	88.1066667	1725	20	34N	10E	4E	640	1996	ELWOOD	LAYNE-WESTERN CO	CS
409787			1709	31	34N	10E	7A	626	1943	MIDEWIN NATIONAL TALLGRASS PRAIRIE - USDA FS	LAYNE-WESTERN CO	IC
311458			1665	30	34N	10E	6A		19990430	VETERANS ADM.-CHG.NATL.CEMETRY	LAYNE-WESTERN/HALL	NC

TABLE 2.4-22D

ISWS PUBLIC GROUNDWATER SUPPLIES WITHIN 25 MILES (Continued)

ISWS WELL ID	COORDINATES		TOTAL DEPTH					GROUND SURFACE ELEVATION	COMPLETION DATE			
	LATITUDE	LONGITUDE	(FEET BGS[1])	SECTION	TOWNSHIP	RANGE	PLOT	(FEET AMSL[2])	(YYYYMMDD)	WELL LOCATION/OWNER	INSTALLED BY	WELL USE
150842			1698	31	34N	10E			19430100	KANKAKEE ORDANCE WORKS	LAYNE WESTERN	IC
412019			1730	09	34N	11E	7A	690	2006	MANHATTAN	LAYNE-WESTERN	CS
410378	41.4472222	87.9975000	1703	17	34N	11E	5D	685	1993	MANHATTAN	L-W CO-WEHLING DIV	CS
400177			1770	21	34N	11E	5F		2000	MANHATTAN	LAYNE-WESTERN	CS

Note: [1] BGS - below ground surface
[2] AMSL - above mean sea level

Well use:
CS - Community Supply
DO - Domestic
IC - Industrial/Commercial
IR - Irrigation
NC - Non-Community Supply
NR - Not Reported
PK – Park
ST - State

All townships and ranges within the 10-mile radius were included within the search.

Therefore, some wells listed may be outside of the 10-mile radius.

Table 2.4-23A

ISGS PRIVATE WATER WELLS WITHIN 2 MILES OF THE BRAIDWOOD SITE

MAP ID	ISWS WELL ID	COORDINATES		TOTAL DEPTH	SECTION	TOWNSHIP	RANGE	GROUND SURFACE ELEVATION	COMPLETION DATE	WELL LOCATION/ OWNER	WELL #	INSTALLED BY	WELL TYPE	STATIC WATER LEVEL	PUMPING INFORMATION		
		LATITUDE	LONGITUDE	(FEET BGS[1])				(FEET AMSL[2])	(YYYYMMDD)					(FT BTOC[3])	PUMPING LEVEL (FT BTOC)	PUMPING RATE (GPM[4])	HOURS PUMPED
1	121970001000	41.263928	-88.218262	1647	8	32	9	577	19370101	Braidwood City	1	Varner C W					
2	121972722600	41.2803	-88.220957	1732	5	32	9	0	19790731	Braidwood, City of	3	Wehling, Richard H.	MUNIC	237	415	1250	8

Note:

[1] BGS - below ground surface
[2] AMSL - above mean sea level
[3] BTOC - below top of casing
[4] GPM - gallons per minute

Deep Water Wells Only

TABLE 2.4-23B

ISWS PRIVATE WATER WELLS WITHIN 2 MILES OF THE BRAIDWOOD STATION

ISWS Well ID	COORDINATES		TOTAL DEPTH	SECTION	TOWNSHIP	RANGE	PLOT	GROUND SURFACE ELEVATION	COMPLETION DATE	WELL LOCATION/ OWNER	INSTALLED BY	WELL USE
	LATITUDE	LONGITUDE	(FEET BGS[1])					(FEET AMSL[2])	(YYYYMMDD)			
409164	41.2641667	88.2191667	1647	08	32N	09E	5C	575	1937	BRAIDWOOD	C VARNER/W L THORNE	
409163	41.2805556	88.2213889	1733	05	32N	09E	6D	560	1979	BRAIDWOOD	WEHLING WELL WORKS	

Note: [1] BGS - below ground surface
[2] AMSL - above mean sea level

Well use:
CS - Community Supply
DO – Domestic
IC - Industrial/Commercial
IR – Irrigation
NC - Non-Community Supply
NR - Not Reported
PK – Park
ST - State

Deep Water Wells Only

Pages 2.4-72 thru 2.4-79 have been intentionally deleted.

TABLE 2.4-24

DESIGN WATER LEVELS FOR SAFETY-RELATED STRUCTURES

SEISMIC CATEGORY I STRUCTURE	FOUNDATION ELEVATION	DESIGN WATER LEVEL (ft)	BASIS FOR DESIGN WATER LEVEL
Reactor Containment	Refer to Figure 3.7-7	600.0	Groundwater
Auxiliary Building	Refer to Figure 3.7-7	600.0	Groundwater
Diesel-generator structures	Refer to Figure 3.7-7	600.0	Groundwater
Fuel Handling Building	Refer to Figure 3.7-7	600.0	Groundwater
Main steam tunnel	Refer to Figure 3.7-7	600.0	Groundwater
Essential service water discharge***	Refer to Figure 3.8-16	590.0	Maximum elevation in essential cooling pond plus wind wave
Pond Screen House* ** ***	568 ft. 3 in.	602.4	PMF at lake plus wind wave

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* Only that portion housing the essential service water intake.

** Pond Screen House is designed for PMF water elevation 598.17 feet plus coincident wave of 4.17.

*** The static and dynamic effects due to nonbreaking waves will be computed based on the procedure given in U.S. Army Corps of Engineers, "Shore Protection Manual."

TABLE 2.4-25

WAVE PARAMETERS APPLICABLE TO THE LAKE SCREEN HOUSE

PARAMETER	NORMAL POOL	NORMAL* POOL	PMF POOL
Lake Water Level (feet)	595	595	598.17
Fetch Length (miles)	0.45	1.25	1.25
Overland Wind Speed (mph)	60	60	40
Significant Wave Height (feet)	2.3	3.0	2.35
Wave Period (seconds)	3.4	2.37	2.39
Average Depth (feet)	11.0	11.46	11.46
Depth at the Structure (feet)	24.8	24.8	28.0
Wave Breaking (B) or Non-Breaking (NB)	NB	NB	NB
Hydrostatic Force (lb/ft)	19,190	19,190	24,461
Height of Line of Action Above Grade (feet)	8.33	8.33	9.33
Hydrodynamic Force (lb/ft)	700	2,260	793
Height of Line of Action Above Grade (feet)	13.92	14.25	15.55

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*Assumed internal dikes not existing.

2.5 GEOLOGY, SEISMOLOGY, AND GEOTECHNICAL ENGINEERING

2.5.1 Basic Geologic and Seismic Data

This section presents basic geologic and seismic data obtained for the Braidwood Station located in Will County, Illinois. The site is about 4.5 miles southwest of the Kankakee River in the northeastern quarter of Section 19 (T.32N., R.9E.). This location is about 1.5 miles southwest of the town of Braidwood and about 22 miles southwest of Joliet. The location of the site in relation to the surrounding area is shown on Figure 2.5-1.

Basic geologic and seismic data for the site were developed by a program of field explorations, laboratory tests, and office studies. This program included the following:

a. research of both published and unpublished geologic and seismic data,

b. geologic reconnaissance of the site and surrounding area,

c. test borings,

d. geophysical explorations,

e. laboratory tests, and

f. excavation mapping.

The purpose of this program was to determine the geology and seismic design parameters for the site. A more detailed program of field explorations was conducted with the specific objective of obtaining foundation design data.

As predicted in the PSAR, the Pennsylvanian age Carbondale Formation and the Pleistocene age Wedron Formation provided a suitable foundation base. The stratigraphy encountered in the plant excavations was the same as that encountered in the PSAR stage borings. No significant unanticipated geologic conditions requiring design changes were encountered in construction.

2.5.1.1 Regional Geology

2.5.1.1.1 Regional Geologic History

2.5.1.1.1.1 General

The study of geologic history provides an insight as to the tectonic stability of a region and a better understanding of stratigraphic relationships between various soil and rock units. It also furnishes correlative data which assist in the interpretation of events in adjacent regions.

An accurate interpretation of geologic history is the result of years of cumulative effort. It is based on numerous examinations of soil and rock units in exposures and from borings with regard to lithology and fossil content. Comparisons are drawn with events observed in present-day environments under the assumption that natural processes have remained constant throughout geologic time.

The plant site is located in part of the glaciated Till Plains Section of the Central Lowland Physiographic Province (Reference 1).

A generalized composite stratigraphic column for north central Illinois is presented on Figure 2.5-2. The entire series of stratigraphic units may not be present at any given locality; however, it is a graphic illustration of the changing geologic history. Individual periods of geologic time are discussed in the following subsections. The ages for the geologic periods are taken from Reference 2.

2.5.1.1.1.2 Precambrian (Earlier Than Approximately 600 Million Years B.P.)

The Precambrian is the oldest recognized division of geologic time, and its history is obscure. In northern Illinois, Precambrian rocks lie some 2000 to 5000 feet below sea level. Their nature and history must be based on boring samples and observations of exposures in more distant regions. In Illinois, 28 borings have reached the Precambrian basement. The borings most commonly encountered medium- to coarse-grained granite. Other rock types reported are quartz monzonite, rhyolite porphyry, and felsite (Reference 3).

In central and northern Wisconsin, Precambrian rocks are exposed at the surface. Here they record alternating periods of sedimentary deposition and erosion, mountain building, metamorphism, and igneous activity. These events were followed by a long period of erosion which reduced the cores of Precambrian mountains and formed a gently sloping peneplain of regional extent. It is inferred that similar events took place throughout northern Illinois.

2.5.1.1.1.3 Cambrian (Approximately 500 to Approximately 600 Million Years B.P.)

Due to the absence of early and middle Cambrian rocks in both northern Illinois and Wisconsin, it is assumed that the long period of erosion which took place in late Precambrian time continued through early and middle Cambrian time. By late Cambrian time, a crustal flexure began to form the Central Interior Basin, and the lowlands were submerged by shallow seas which advanced from the south. Several thousand feet of sediments, now comprising the Croixan Series, were deposited in northern Illinois. Conditions were variable from time to time, producing a wide variation of rock type. Cambrian time was ended by uplift of the region above sea level succeeded by a period of erosion.

The area of central Wisconsin was probably uplifted several times during the Paleozoic Era. Initial movements may have taken place during Cambrian time to form the Wisconsin Dome, but evidence for time and spatial relations is scarce (Reference 4).

2.5.1.1.1.4 Ordovician (430 ±10 to Approximately 500 Million Years B.P.)

The Ordovician period began with a readvance of the sea. Initial deposition consisted primarily of calcareous materials followed by alternating periods of clastic and calcareous deposition, all of which comprise the Canadian Series.

At the end of Canadian time, regional uplifting occurred, and widespread erosion was initiated. A well-defined river system was developed in portions of northern Illinois. Deep erosional valleys were produced, which possibly cut into the underlying Cambrian sediments. During this period, significant uplift of the Wisconsin Dome is recognized.

Following the long period of erosion, Champlainian time was initiated by an advance of the sea over existing erosional topography. Unconsolidated soils were reworked, and sand was deposited under conditions which remained stable over a vast area. The period of sand deposition was followed by relatively brief emergence. Again the sea advanced to initiate some reworking, after which a long period of calcareous deposition in clear seas with subsequent emergence completed the Champlainian Series.

Emergence was apparently brief, and the region was again submerged by a sea which advanced from the south. Deposits of the Cincinnatian Series initially consisted of thick accumulations of silt and mud, probably in shallow, turbid seas. Later, the seas cleared, and calcareous deposits were formed. Finally, turbid conditions returned with the deposition of more silt and mud.

The Kankakee Arch may have begun to form during the Ordovician. Structural relief is believed to have occurred by subsidence of the Illinois and Michigan Basins while the Arch remained stable, forming a boundary between them. This stability has been theoretically attributed to the presence of an underlying eroded core of a Precambrian mountain range.

Ordovician time was ended by elevation of the region above sea level followed by long continued erosion.

2.5.1.1.1.5 Silurian (400 ±10 to 430 ±10 Million Years B.P.)

After an erosional interval of long duration, northern Illinois was again inundated during Alexandrian time by marine waters that advanced from the Gulf of Mexico and eventually connected with seas to the north. Deposition began with clastic sediments in shallow seas which became clear before the end of Alexandrian time, as indicated by pure carbonates.

Deposition during the following Niagaran time is represented by a thick carbonate sequence characterized by reefs. This period of deposition was followed by uplift and then erosion which began late in Silurian and continued into Devonian time.

2.5.1.1.1.6 Devonian (340 ±10 to 400 ±10 Million Years B.P.)

Following the deposition of the Silurian beds, a second uplifting of the Wisconsin Dome is recognized. This time, an arch was formed that extended southeastward from Wisconsin into Illinois, almost to the city of Kankakee. This structure is called the Wisconsin Arch.

Erosion in northern Illinois continued from Silurian through early Devonian time. The middle Devonian deposition began with a major transgression of the sea. The Sangamon and Kankakee Arches acted as barriers for a time. Sedimentation began with an accumulation of calcareous materials and ended with the emergence of the region above sea level and subsequent erosion, which appears to have removed much of the Devonian rocks in northern Illinois.

2.5.1.1.1.7 Mississippian (320 ±10 to 340 ±10 Million Years B.P.)

The events which took place in northern Illinois between the close of the Devonian and the beginning of the Pennsylvanian period are not clearly known.

Mississippian-aged deposits are generally not found in extreme northern Illinois. It is postulated that Mississippian seas never advanced completely over the region. Where observed, however, the deposits consisted predominantly of marine carbonates. Initial deposits, however, were silt and mud. A somewhat gradual transition is indicated in adjacent regions between early Mississippian and late Devonian deposition.

During this span of time, the major folding of the La Salle Anticline took place, probably during late Mississippian, along with additional movement of the Kankakee Arch. The Sandwich Fault was probably formed as a result of subsequent relaxational movements. These tectonic movements were accompanied by widespread erosion which removed existing Mississippian- and Devonian-aged deposits throughout most of northern Illinois. In

Wisconsin, the Wisconsin Dome had become prominent by the close of Mississippian time.

2.5.1.1.1.8 Pennsylvanian (270 ± 5 to 320 ± 10 Million Years B.P.)

In Pennsylvanian time, conditions controlling sedimentation were considerably different from those in which earlier Paleozoic sediments were deposited. Throughout Pennsylvanian time, highland areas existed along the eastern and southern parts of North America. The interior part of the continent was a plain that was repeatedly submerged by the sea or lay a short distance above it. When the sea submerged the plain, streams from the highland areas carried rock debris into it. The resulting deposition accumulated in a marine environment. As the sea receded, deposition continued, but the deposits accumulated in a terrestrial environment. The newly emerged plain became covered by swamps which extended unbroken for hundreds of miles. Vegetation flourished and accumulated in thick deposits to form the coal layers which exist today. Eventually the sea returned to initiate another cycle of sedimentation. Each cycle is therefore partly marine and partly terrestrial in origin. Numerous similar cycles of deposition, many of which are separated by localized erosional unconformities, have been recorded in the Pennsylvanian stratigraphy.

Pennsylvanian deposits thin over the La Salle Anticline, indicating some continued tectonic movement. Late or post-Pennsylvanian time marked the climax of the La Salle Anticlinal folding. Probably all the structural units involved in the late Mississippian period of folding were again affected.

2.5.1.1.1.9 Permian (225 ±5 to 270 ±5 Million Years B.P.)

No deposits of Permian age have been found in the regional area. The apparent absence of deposits indicates that the Permian was a period of nondeposition or that Permian deposits in the area were subsequently eroded.

2.5.1.1.1.10 Triassic (190 ±5 to 225 ±5 Million Years B.P.)

There are no deposits of Triassic age in the regional area. This is largely a period of erosion (Reference 5).

2.5.1.1.1.11 Jurassic (135 ±5 to 190 ±5 Million Years B.P.)

There are no deposits of Jurassic age in the regional area. This was largely a period of erosion (Reference 5).

2.5.1.1.1.12 Cretaceous (65 ±2 to 135 ±5 Million Years B.P.)

Cretaceous deposits are not encountered in northern Illinois. It seems likely that Cretaceous seas did not advance much beyond western central Illinois, where relatively small areas of Cretaceous rocks are known to be present.

Geologic evidence suggests that northern Illinois existed as a low, stable land mass for about 200,000,000 years, while the Appalachian Mountains, Rocky Mountains, and other structural features in North America were being formed or undergoing additional movements.

2.5.1.1.1.13 Quaternary (Present to 2 ± 1 Million Years B.P.)

Glaciation in northern Illinois began with the Pleistocene some one million years ago. Four major glacial advances invaded the region as illustrated on Figure 2.5-3 (Reference 6).

The ice fronts probably advanced and retreated several times during each of these glacial periods, but the record is scant for the Kansan and absent for the Nebraskan, the oldest advance. The areas covered by ice during each of the advances, even from the same sources, were not identical. The result is a complex series of partially overlapping deposits and associated features, each of which was modified, to a greater or lesser degree, by subsequent events.

During each advance, the glaciers eroded preexisting deposits. Debris was also deposited from the ice in the form of till plains, moraines, and outwash during the advance and retreat of the ice. Meltwater flowing away from the glacier front was also responsible for eroding, reworking, and redepositing many of these materials. Windblown silt, derived from the outwash sediments in the valleys of the meltwater streams, was widely distributed over the land surface well beyond the glacier front. Sand dunes were also developed locally.

Between glaciations, the climate returned to more temperate conditions. Streams developed their drainage systems; at least initially, their positions were controlled largely by the character of the surface left by the retreating glaciers. As these materials were exposed, weathering processes began modifying them. The thickness and character of the resulting soils are largely functions of climate, topographic position, vegetation, and duration of the interglacial age.

Deposits of the Wisconsinan glacial advance are relatively well preserved, since the Wisconsinan was the last of the great ice sheets to invade northern Illinois. Many of the present-day land forms are attributable to this last advance.

2.5.1.1.2 Physiography

The northern portion of the midwestern United States is located in the Central Lowland Physiographic Province. This physiographic province has been divided into several physiographic sections. Parts of northern Illinois (Figure 2.5-4) are located in the Wisconsin Driftless Section, the Till Plains Section, and the Great Lakes Section of the Central Lowland Physiographic Province.

The site is located near the eastern margin of the Till Plains Section. The Till Plains Section is characterized, in general, by the presence of glacial deposits overlying the bedrock surface. Local outcrops of bedrock are present. The Till Plains Section in Illinois is further subdivided into the following physiographic subsections: the Rock River Hill Country, the Green River Lowland, the Bloomington Ridged Plain, the Galesburg Plain, the Kankakee Plain, and the Springfield Plain.

The site area is located in the Kankakee Plain physiographic subsection. This subsection is characterized in the northeastern portion by gently rolling topography formed by glacial deposits, and in the remaining portions by essentially flat-lying topography representing former glacial lakes. The northwest-trending Kankakee River and the southwest-trending Illinois River pass through the subsection. Bedrock is locally exposed throughout the area.

2.5.1.1.3 Stratigraphy

2.5.1.1.3.1 Soil Units

The soil in the region adjacent to and within the plant site area consists of eolian deposits, lacustrine deposits, outwash, and glacial till of the Wisconsinan glacial stage, and some residual soil formed in the upper part of the Pennsylvanian bedrock. In the borings at the site, the total thickness of these deposits ranges from 26.0 to 62.0 feet (see Subsection 2.5.1.2.4.1). Present locally in the region are deposits of alluvium, loess, and strip mine debris.

The surficial eolian deposits of the Parkland Sand (see Subsection 2.5.1.2.4.1.1.1) consist primarily of silty fine sands and occur as sand dunes and sheet-like deposits (Reference 6).

The lacustrine deposits of the Dolton Member of the Equality Formation (see Subsection 2.5.1.2.4.1.1.2) are mainly sands with some silt. Some localized deposits of gravel and pebbly sand occur, which are thought to be the result of wave erosion of till and ice front deltas.

The till has been classified as part of the Wedron Formation (see Subsection 2.5.1.2.4.1.1.3). The Wedron Formation is predominantly till, but also contains interbedded outwash gravel, sand, and silt (Reference 6).

Underlying the soil units locally are areas of highly weathered Pennsylvanian bedrock or residual soil.

2.5.1.1.3.2 Rock Units

The distribution of the rock units which form the bedrock surface within a broad region is shown on Figure 2.5-5. The rock units include a sedimentary sequence of Cretaceous-, Pennsylvanian-, Mississippian-, Devonian-, Silurian-, Ordovician-, and Cambrian-aged strata and an igneous and metamorphic complex of Precambrian-aged rocks as shown on Figures 2.5-6 and 2.5-7.

The sedimentary rock sequence of northern Illinois in the proximity of the site includes Pennsylvanian-, Silurian-, Ordovician-, and Cambrian-aged strata. These strata consist of approximately 5000 feet of limestones, dolomites, sandstones, coals, and shales which rest on the Precambrian basement. The basement consists of granites and granodiorites (Reference 3). The relationships of these rock units to each other are shown on Figure 2.5-2.

2.5.1.1.4 Structures

The plant site lies within a tectonic province of North America called the Central Stable Region, which is characterized by a sequence of southward-thickening Paleozoic strata overlying the Precambrian basement. During Paleozoic and early Mesozoic times, this area was subjected to a series of vertical crustal movements which formed broad basins and intervening arches. The basins and arches have been modified by local folding and faulting. Major geologic structures are shown on Figures 2.5-8 and 2.5-9.

2.5.1.1.4.1 Folding

The distribution of major folds in the region is shown on Figure 2.5-8 and their characteristics are presented in Table 2.5-1. The site area is located on the west side of the approximately northwest-southeast-trending Kankakee Arch; it is east of the northwest-southeast-trending La Salle Anticlinal Belt. The knowledge of these structural features is based on surface and/or subsurface geological data. The geologic age of the most recent movement associated with these major structural features is considered to be pre-Cretaceous, with the major movement occurring in Paleozoic time.

The direction and amount of regional dip of the strata in northern Illinois vary. In the vicinity of the site area, the regional dip is gently toward the Illinois Basin.

2.5.1.1.4.1.1 Illinois Basin

The Illinois Basin is oval shaped. It has a major axis which trends approximately N 25° W and is approximately 350 miles long, and a minor axis which is approximately 250 miles long. The deepest part of the basin is in southeastern Illinois.

To the north, the Illinois Basin rises gently to the Wisconsin Arch. To the northeast, the Illinois Basin is separated from the Michigan Basin by the Kankakee Arch. To the east, the Illinois Basin rises gently to the Cincinnati Arch (which is outside of the regional area). To the south, the Illinois Basin rises gently to the Pascola Arch (which is Outside of the regional area). To the southwest, the Illinois Basin is bordered by the Ozark uplift (which is also outside of the regional area). To the west, the Illinois Basin rises gently to the Mississippi River Arch (Reference 7).

The Illinois Basin began to form in the Cambrian and continued to develop intermittently until the end of the Pennsylvanian (Reference 8). The depositional center of the basin migrated throughout this time span.

2.5.1.1.4.1.2 Wisconsin Arch and Kankakee Arch

The Wisconsin Arch is a south-southeast-trending extension of the Wisconsin Dome (Figure 2.5-8). It can be traced into Illinois to the vicinity of the city of Kankakee, where it appears to connect with the Kankakee Arch of Illinois and Indiana (Figure 2.5-8). The Wisconsin Arch has a Precambrian core and is believed to be the result of crustal uplift, whereas the Kankakee Arch acquired its structural relief chiefly by greater subsidence of the structural basins which lie on either side of the arch (Reference 4).

2.5.1.1.4.1.3 LaSalle Anticlinal Belt

The LaSalle Anticlinal Belt represents a major Paleozoic structural feature in north central and eastern Illinois. Included within the term "LaSalle Anticlinal Belt" are a number of subsidiary structures, including the Ashton Arch, Oregon Anticline, Downs Anticline, etc. (Reference 9). Whether all of these structures are genetically interrelated, however, remains a matter of some discussion among workers in the field.

Movement on the LaSalle Anticlinal Belt took place over a significant portion of mid- to late Paleozoic time. In general, earlier movement occurred in the northern part of the belt, with progressively younger movement occurring southward along the belt (References 9 and 10).

2.5.1.1.4.1.4 Ashton Arch

The Ashton Arch is a broad anticline located on the southern side of the Sandwich Fault at the northern end of the LaSalle Anticline. McGinnis (Reference 11) and Green (Reference 12) interpreted the Ashton Arch as being a horst (uplifted fault block) and referred to it as the Ottawa Horst.

Postulation of a relationship between the LaSalle Anticlinal Belt and the Ashton Arch appears to be reasonable in view of the similarity in the age of movement on the structures involved as well as the en echelon relationship of the various structural elements of the La Salle Anticlinal Belt as suggested by Clegg (Reference 9).

2.5.1.1.4.1.5 Herscher Dome

The Herscher Dome is located approximately 10 miles southeast of the site (see Figures 2.5-10 and 2.5-12). It is an asymmetrical anticlinal structure about 3 miles wide east-west and 5 miles long north-south, with over 150 feet of closure. As in other en echelon structures in the LaSalle Anticlinal Belt, the strata dip rather steeply on the southwest flank of the Herscher Dome and more gently on the northeast side (Reference 13).

2.5.1.1.4.1.6 Downs Anticline

About 60 miles to the southwest of the site is a small flexure trending parallel to the LaSalle Anticlinal Belt and known as the Downs Anticline.

2.5.1.1.4.1.7 Mattoon Anticline

The Mattoon Anticline trends roughly north-south and is located approximately 110 miles southeast of the site.

2.5.1.1.4.1.8 Tuscola Anticline

The Tuscola Anticline is one of the many subsidiary structures of the LaSalle Anticlinal Belt (Reference 14). It extends south-southeastward from north of Tuscola in Douglas County to near Charleston in Coles County, Illinois. The anticline plunges southeastward and is broader at the north than at the south. At its closest point, it is approximately 90 miles south of the site.

2.5.1.1.4.1.9 Murdock Syncline

The Murdock Syncline is east of the Tuscola Anticline and shares a common flank with it (Reference 14). The exact extent of the structure is unknown. It probably dies out to the north in Champaign County, Illinois, approximately 100 miles from the site (Reference 14). To the south it can be traced only to the vicinity of Charleston, Coles County, Illinois (Reference 14).

2.5.1.1.4.1.10 Marshall Syncline

The Marshall Syncline trends approximately north-south and is located approximately 100 miles from the site. It is an asymmetrical fold with a comparatively steep west flank (Reference 14).

2.5.1.1.4.1.11 Folded Structures Associated with the Plum River Fault Zone

Four minor structures are associated with the Plum River Fault Zone. Located successively from west and east along the fault zone, they are the Uptons Cave Syncline, the Forreston and Brookville Domes, and the Leaf River Anticline (Figure 2.5-11).

The Forreston and Brookville Domes were previously considered to be a single domal structure called the Brookville Dome until subsequent drilling indicated the presence of two domal structures. All four of these minor structures are considered to be associated with the development of the Plum River Fault Zone (Reference 15).

2.5.1.1.4.1.12 Louden Anticline

The Louden Anticline is located approximately 150 miles south of the site. It trends north-south and extends from the northern county line of Marion County through east-central Fayette County, Illinois. It is approximately 19 miles long.

2.5.1.1.4.1.13 Salem Anticline

The Salem Anticline trends approximately parallel to the Louden Anticline, and extends from central Jefferson County, Illinois, to central Marion County, Illinois. It is approximately 25 miles long.

2.5.1.1.4.1.14 Clay City Anticline

The Clay City Anticline is located approximately 150 miles from the site. It trends north-south from northeastern Hamilton County through Wayne County, Illinois, where it bends and trends N 27° E through Clay, Richland, and Jasper Counties, Illinois. The axial trace of the Clay City Anticline is approximately 57 miles long. The anticline is a semicontinuous series of anticlinal uplifts separated by saddles (Reference 16). Du Bois and Siever (Reference 16) noted that the amplitude of the anticline increases with depth and decreases in the overlying Pennsylvanian strata. They interpreted this to imply that the structure developed during pre-Pennsylvanian time; however, the presence of the fold in the Pennsylvanian strata indicates some folding was Pennsylvanian and/or post-Pennsylvanian.

2.5.1.1.4.1.15 DuQuoin Monocline

Located approximately 180 miles south of the site, the DuQuoin Monocline is a steep eastward-dipping monoclinal structure that trends north-south from northernmost Jackson County through Perry, Jefferson, and Marion Counties, Illinois. The DuQuoin Monocline is 48 miles long and separates the deepest part of the Illinois Basin, the Fairfield Basin, from the shallower western portion of the basin. Pennsylvanian strata east of the monocline are thicker than equivalent beds to the west (Reference 17). The monocline is broken by subordinate faults (Reference 10). Flexure of the DuQuoin Monocline is considered to have begun in the Late Mississippian and was completed by the Middle Pennsylvanian (Reference 17).

2.5.1.1.4.1.16 Mississippi River Arch

The Mississippi River Arch is a broad arch trending roughly parallel to the Mississippi River approximately 150 miles west of the site (Figure 2.5-8).

2.5.1.1.4.1.17 Pittsfield and Lincoln Anticlines

Located approximately 180 to 210 miles southwest of the site and near the Mississippi River are the Pittsfield and Lincoln Anticlines. These two folds parallel each other and trend northwest-southeast.

2.5.1.1.4.1.18 Mineral Point and Meekers Grove Anticlines

The Mineral Point Anticline and Meekers Grove Anticline are located in southwest Wisconsin, approximately 150 miles northwest of the site and trend roughly east-west.

2.5.1.1.4.1.19 Baraboo, Fond du Lac, and Waterloo Synclines

Also located in southern Wisconsin are three synclinal structures, the Baraboo Syncline, Fond du Lac Syncline, and Waterloo Syncline. These synclines trend east-west to northeast-southwest and are located about 140 to 170 miles north of the site.

2.5.1.1.4.1.20 Leesville Anticline

The Leesville Anticline is a structure that trends approximately N 15° W and extends from southeastern Lawrence to northern Monroe Counties in south central Indiana (Figure 2.5-8). The Leesville Anticline is a major anticlinal structure composed of five domes in an approximate northwest-southeast alignment. The anticlinal structure lies approximately 1 to 2 miles west of, and parallel to, the Mt. Carmel Fault. Between the fault and the anticline is a series of narrow synclines that close against the fault (Reference 18).

Melhorn and Smith (Reference 18) consider the disturbance along the Leesville Anticline and Mt. Carmel Fault to be genetically related to the La Salle Anticlinal Belt. Deformation along the Leesville Anticline is therefore Late Mississippian and pre-Mesozoic (Reference 18).

2.5.1.1.4.1.21 Michigan Basin

The Michigan Basin is a roughly circular structural basin located in Michigan, northwestern Ohio, western Ontario, northeastern Illinois, and eastern Wisconsin. The basin is bordered on the southwest by the Kankakee Arch, on the south by the Indiana-Ohio Platform (not shown on Figure 2.5-8) on the southeast and east by the Findlay Arch and Algonquin Arch (not shown on Figure 2.5-8), and on the west by the Wisconsin Arch. The northern portion of the basin rises gently to the Precambrian rocks of the Canadian Shield. The basin is herein defined on the -1000-foot contour on top of the Trenton Limestone of Ordovician age (Reference 19). Structure contours on the top of the Trenton Limestone indicate that the strata dip into the deepest part of the basin at approximately 60 ft/mi (approximately 0.65°). The deepest part of the basin is located just west of Saginaw Bay, Michigan, where approximately 14,000 feet of sediments overlie the Precambrian basement rocks (Reference 19). The Michigan Basin began to develop during the Late Cambrian and continued as a negative structural feature until the Middle Pennsylvanian. There was additional accumulation of some sediment in the Michigan Basin outside the regional area during Jurassic time (Reference 19).

2.5.1.1.4.1.22 Structural Contour Maps

Regional structural contours on top of the Galena Group are shown on Figure 2.5-12. This map shows the detail of some of the structural features in northern Illinois. From this figure it is seen that the western end of the Ashton Arch and the western flank of the Oregon Anticline dip into the Polo Basin.

No published regional structure contour maps on top of the Prairie du Chien, Kankakee, or Carbondale formations have been located. The Illinois State Geological Survey has indicated that contouring the top of the Prairie du Chien, while possible, is of limited value because erosion cut deeply into the Prairie du Chien prior to deposition of the overlying St. Peter Sandstone. Therefore, since the surface of the Prairie du Chien is due to erosion, surface contours would not be representative of its structure.

Difficulty is encountered in identifying the top of the Kankakee and, as a result, attempts to construct a structure contour map on top of the Kankakee have not met with significant success. Furthermore, regional correlations have shown that efforts to contour the top of the Kankakee have resulted in maps very similar to structural contour maps drawn on top of the Galena Group.

Contouring the top of the Carbondale has not been productive because, with the exception of the Colchester (No. 2 Coal) member, the Carbondale is not marked by any consistently mappable units.

2.5.1.1.4.2 Faulting

The distribution of major faults in the region is shown on Figure 2.5-9, and their characteristics are presented in Table 2.5-2. The Sandwich Fault Zone is the major fault closest to the site area.

2.5.1.1.4.2.1 Sandwich Fault Zone and Plum River Fault Zone

The Sandwich Fault Zone trends northwest through northern Illinois. It is mapped on the surface and in the subsurface for a distance of approximately 85 miles. It is an essentially vertical fault with a maximum displacement of approximately 900 feet (Reference 20). The northeastern side has moved down relative to the southwestern side. Movements along the fault zone occurred in the interval between post-Silurian and pre-pleistocene time. No rocks of intervening ages are present, which prevents better definition of the movements. However, major movements along the fault zone may have been contemporaneous with folding of the La Salle Anticlinal Belt (Reference 21) during the late Paleozoic.

The Plum River Fault Zone (formerly the Savanna Fault and the Savanna Anticline), located approximately 100 miles northwest of the site, is a generally east-west-trending zone of high-angle, possibly en echelon faults extending from Leaf River (Ogle County), Illinois, to southwest of Maquoketa (Jackson County), Iowa (Reference 15). The fault zone is less than 0.5 mile wide. Vertical displacement along the fault is 100 to 400 feet, with the north side downthrown. The age of movement has been limited to post-middle Silurian to post-Pennsylvanian (Reference 15). The fault zone is overlain by unfaulted Pleistocene deposits. Four minor structural features are associated with the fault zone: the Forreston Dome, the Brookville Dome, the Leaf River Anticline, and the Uptons Cave Syncline.

2.5.1.1.4.2.2 Chicago Area Faults

2.5.1.1.4.2.2.1 Chicago Area Basement Fault Zone

On the basis of gravity and seismic geophysical evidence, McGinnis (Reference 22) postulated a basement fault zone in the metropolitan Chicago area, north of and about parallel to the Sandwich Fault Zone. The presence of the fault has not been verified.

2.5.1.1.4.2.2.2 Chicago Area Minor Faults

As a result of a recent seismic survey in the metropolitan Chicago area, 25 faults were reported with inferred displacement up to 50 feet (Reference 23). None of these involves wide shear zones or detectable scarps on the rock surface. Faults that have been observed in natural outcrops and quarries in the Chicago area have displacements from a few inches to a few feet, but most are less than 1 foot (Reference 23). These faults are not the same as those inferred from the geophysical survey.

2.5.1.1.4.2.3 Oglesby and Tuscola Faults

The Oglesby Fault and the Tuscola Fault, postulated by Green (Reference 12), occur on the western flank of the La Salle Anticline (Figure 2.5-9). Studies by the Illinois State Geological Survey have indicated that the areas where the faults are postulated have dips steeper than the regional dip. No evidence has been found confirming major faulting along the trends of the postulated Oglesby and Tuscola Faults (Reference 24).

2.5.1.1.4.2.4 Centralia Fault

The Centralia Fault is a series of several north-south-trending faults in Marion and Jefferson Counties, Illinois. The faults have no surface expression and are known only from subsurface data, primarily mine records (Reference 25). The faulted zone is approximately 20 miles long and displays a maximum displacement of 200 feet, downthrown to the west. Faulting is believed to be post-Pennsylvanian but pre-Cretaceous in age (Reference 17).

2.5.1.1.4.2.5 Cap Au Gres Faulted Flexure

Southwest of the plant site about 200 miles lies the Cap Au Gres Faulted Flexure, which extends continuously from western Pike County, Missouri, southeastward toward Lincoln County, then east across southern Calhoun County, Illinois, and into southwestern Jersey County, where it disappears beneath the broad alluvial valley of the Mississippi River. Throughout its length, the flexure is a narrow zone along which the rocks dip steeply, southward or southwestward. The total uplift or "structural relief" along the flexure averages about 1000 feet, but it varies from place to place (Reference 26). Deep faulting has been inferred on the basis of steep dips, although the surface strata do not appear to be faulted. The principal folding of the Cap Au Gres Faulted Flexure was post-St. Louis (Mississippian) and pre-Pottsville (Pennsylvanian) (Reference 26). Later periods of movement may have occurred; however, no rocks younger than Paleozoic are present with which to date possible displacements.

2.5.1.1.4.2.6 Mifflin Fault

The Mifflin Fault is located in Iowa and Lafayette Counties, Wisconsin, approximately 160 miles northwest of the site. The fault trace is approximately 10 miles long with a strike of N 40° W (Reference 27). The southwest side of the fault is downdropped at least 65 feet, and there is about 1000 feet of strike-slip displacement (Reference 28). The last movement on the fault is believed to be late Paleozoic (Reference 28).

2.5.1.1.4.2.7 Postulated Wisconsin Faults

Thwaites' map of the buried Precambrian surface in Wisconsin (Reference 29) postulated the existence of four faults in the southern and eastern sections of the state. For convenience, these have been named the Janesville, Appleton, Waukesha, and Madison Faults. Ostrom (Reference 30) stated that Thwaites' map is diagrammatic and does not represent detailed study of each fault. He reported that differences in elevation of the basement, interpreted by Thwaites to be the result of faulting, are now believed to be due to topographic relief on the erosional basement surface.

The Green Bay Fault, also described by Thwaites (Reference 29), is located about 250 miles north of the site. The trend of the fault is northeast-southwest. It is believed to be a reef structure in Silurian rocks (Reference 30).

2.5.1.1.4.2.8 Mt. Carmel Fault

The Mt. Carmel Fault trends north-northwest for a distance of about 50 miles in south central Indiana. It is a normal dip-slip fault that dips about 69° west and has about 80 to 175 feet of vertical displacement. Movement of the fault may have begun in late Mississippian and probably was concluded by early Pennsylvanian time (Reference 18).

2.5.1.1.4.2.9 Royal Center Fault

The Royal Center Fault, which is located in northern Indiana (Figure 2.5-9), trends northeast-southwest for about 47 miles, with the southeast side downthrown about 100 feet relative to the northwest side (Reference 31).

2.5.1.1.4.2.10 Fortville Fault

The Fortville Fault trends north-northeast to south-southwest for about 55 miles through central Indiana (Figure 2.5-9). The southeast side of the fault is downthrown about 60 feet relative to the northwest (Reference 31).

2.5.1.1.4.2.11 Cryptovolcanic or Astrobleme Structures

In addition to the areas of faulting, there are four cryptovolcanic or astrobleme structures within the regional area and its immediate periphery (Figure 2.5-9). These structures include: the Des Plaines Disturbance, the Kentland Disturbance, the Glovers Bluff Disturbance, and the Glasford Disturbance. These four structures, which range from about 0.5 mile to 5 miles in diameter (References 4, 32, and 33), are all probably of Late Paleozoic or Mesozoic Age.

2.5.1.1.4.2.12 Faults Beyond 200 Miles from the Braidwood Site

Beyond 200 miles, but of importance to the regional geology, are the fault zones in the southern Illinois area.

2.5.1.1.4.2.12.1 Rough Creek Fault Zone

The Rough Creek Fault Zone trends east-west across southern Illinois into Kentucky. The western portion of this fault zone has previously been referred to in the literature as the Cottage Grove Fault Zone, and the eastern portion has previously been referred to as the Shawneetown Fault Zone. Good exposures along the fault zone are rare, and most interpretations are based on subsurface data. According to Bristol and Buschbach (Reference 7) and Sutton (Reference 34), the fault zone consists at some localities of a series of high-angle reverse faults with the south side being the upthrown side, and at other localities of a series of normal, block faults. Summerson (Reference 35) and Heyl (Reference 36) suggest strike-slip or wrench-type movement for this system. Heyl states that the numerous horsts and grabens are typical of wrench-type faults. Details of the faulting in the Rough Creek area are shown in Weller et al. (Reference 37); Stonehouse and Wilson (Reference 38); Heyl (Reference 36); and in References 39 and 40.

2.5.1.1.4.2.12.2 Structural Relations of Faults North and South of the Rough Creek Fault Zone (Including the Wabash Valley Fault Zone)

Faulting is present both north and south of the Rough Creek Fault Zone. The faults on the north side strike northeast and those along the Illinois-Indiana border are collectively referred to as the Wabash Valley Fault Zone. Those northeast-trending faults, including the Wabash Valley Fault Zone, are high angle faults with maximum displacements within the magnitude of 200 to 300 feet. The location and northern extent of these faults is well defined on the basis of boring data. These faults terminate at the Rough Creek Fault Zone and are offset from the traces of the faults on the south side of the Rough Creek Fault Zone.

The faults on the south side of the Rough Creek Fault Zone trend northeast to southwest and east to west (Reference 41). The number of faults and the amount of their displacement is much greater on the south side than on the north side. Displacements in excess of 1500 feet have been reported by Eardley (Reference 4). The faults on the south side which intersect the Rough Creek Fault Zone also terminate at this zone, as do the faults on the north side.

A fault zone along the Mississippi Valley is located in southern Illinois and in adjoining states, approximately 250 miles south of the plant site. In southern Illinois, this structure consists of a series of northeast-striking faults. The geological and geophysical evidence suggests that the Mississippi Valley Fault

Zone is associated with the Mississippi Embayment tectonic element.

2.5.1.1.4.2.12.3 Ste. Genevieve Fault Zone

At the western end of the Rough Creek Fault Zone there is a series of northwest-trending faults along the border between southern Illinois and southwest Missouri, several of which are grouped as the Ste. Genevieve Fault Zone (Figure 2.5-9). These faults are generally high-angle faults along which the north side has moved down relative to the south side. Displacements of 1000 to 2000 feet have been reported (Reference 21).

2.5.1.1.4.2.12.4 Age of Faulting in Southern Illinois and Adjacent Areas

The age of the faulting in southern Illinois, southern Indiana, northern Kentucky, and southeastern Missouri is post-Pennsylvanian-pre-Pleistocene (Reference 21). In southern Illinois and northern Kentucky the Cretaceous-aged deposits are not generally cut by faults, and it is possible that all the faulting in this area is pre-Cretaceous in age. Willman (Reference 42) and References 39 and 40 show unfaulted Cretaceous deposits overlying the faults on the south side of the Rough Creek Fault Zone.

Ross (Reference 43) states that faults in the upper portion of the Gulf Coastal Embayment area may still be active. This was discussed at great length with members of the Illinois State Gelogical Survey, Indiana Geological Survey, and U. S. Geological Survey. The conclusions are that there is no evidence of displacement of Pleistocene deposits associated with the Rough Creek Fault Zone or with the faults on either side of this zone and that there is no evidence of displacement of the Cretaceous sediments in southern Illinois or northern Kentucky. It was further stated that in Illinois, the term "active" has been loosely applied to faults indicating areas of seismic activity rather than surface movements along a fault plane. Grohskopf (Reference 44) states that faulting of Pliocene gravels and Pleistocene loess is present in southeastern Missouri. However, the upper Cretaceous rocks of the Gulf Coastal Embayment area clearly overlap the intensely faulted area in southern Illinois, and only a few minor faults, possibly the result of slumping or solution collapse, have been found cutting the Cretaceous, Tertiary, or Pleistocene rocks (Reference 5).

2.5.1.1.5 Gravity and Magnetic Anomalies

Measurements of the earth's gravitational and magnetic fields have been made both on the ground and from the air in Illinois and surrounding areas.

Gravity anomalies are usually caused by some combination of the following three major factors:

a. structure, nonconformities, and lithologic changes in the sedimentary rocks;

b. relief on the crystalline basement surface; and

c. lateral density changes in the crystalline portion of the earth's crust and upper mantles (Reference 45).

The gravity field in the region of the site is representative of that in the continental interior, where the tectonics and structural development of the Precambrian crust have been inactive since late Precambrian time. Gravity anomalies are caused primarily by mass differences below the Precambrian surface, although a minor portion of the field is contributed by structures within the Paleozoic section.

Precambrian anomalies are caused by plutons of batholithic proportions, the tops of which are truncated at the Precambrian surface. The bottoms of the plutons are located near the base of the crust at a depth of nearly 35 kilometers (Reference 46).

Figure 2.5-13 represents the Bouguer gravity anomaly map of the region surrounding the site. Some of the anomalies appear to be associated with some of the regional geological structures.

An anomalous trend in east central Illinois appears to follow the trend of the LaSalle Anticlinal Belt. In south central Illinois, a gravity anomaly appears to coincide with the position of the Illinois Basin. In northwest Illinois, a general east-west-trending anomaly appears to follow the trend of the Plum River Fault Zone; and in north central Illinois, an anomaly appears to coincide with the position of the Ashton Arch, on the southwest side of the Sandwich Fault. The relation of gravity anomalies to other geologic structures is not apparent on the regional basis.

The regional aeromagnetic map is shown in Figure 2.5-14. It shows a magnetic anomaly on the south side of the Sandwich Fault, suggesting the basement rock is closer to the surface than in adjacent areas or that there is a change in the magnetic susceptibility of the rocks. The trends of the Wisconsin Arch, the La Salle Anticlinal Belt, and the Kankakee Arch are weakly delineated.

On the basis of gravimetric and seismic geophysical evidence, McGinnis (Reference 22) postulated a basement fault zone in the metropolitan Chicago area, north of and about parallel to the Sandwich Fault Zone. The presence of the fault has not been verified.

2.5.1.1.6 Man's Activities

For a discussion of man's activities in the site area, refer to Subsection 2.5.1.2.7.

2.5.1.2 Site Geology

2.5.1.2.1 General

The site is located within a flat-lying, glacial lake plain area in southwestern Will County near the towns of Braidwood and Godley, Illinois. Elevations of the natural land surface within the site area range from approximately 580 to 610 feet. Strip mining for coal has significantly altered the topography over large areas (see Figure 2.5-15), with vertical cuts approaching 100 feet. In addition, low mounds have been formed at various localities by refuse dumps from underground coal-mining activity.

During this period of investigation, 117 borings and 13 test pits were made throughout the site area at the locations indicated on Figure 2.5-16. A maximum of 15,460 feet of soil and rock were logged and sampled. The maximum depth penetrated was 345.8 feet in Boring L-4. It was completed at an elevation 234.8 feet above sea level, the lowest horizon reached. A water well was drilled in September 1974 to provide groundwater for construction supply (see Figure 2.5-16, Sheet 3, and FSAR Attachment 2.5C). Final depth of the water well was 1753 feet.

In addition to drilling and sampling the various soil and rock units which underlie the site, geophysical logging of several borings was accomplished to verify the position of stratigraphic horizons. The site was inspected by geotechnical staff from Sargent & Lundy and Dames & Moore. Exposures of soil and bedrock were examined as to lithology, weathering characteristics, stratigraphy, and structure. Available geologic literature concerning the site area was researched, and various recognized authorities from the Illinois State Geological Survey were consulted personally and by telephone.

Geologists from Sargent & Lundy mapped and photographed the exposed soil and rock in the excavation for Units 1 and 2 (see Subsection 2.5.4.3.1.1). Members of the Illinois State Geological Survey visited the site to check the stratigraphic descriptions and interpretations delineated during the mapping program (see FSAR Attachment 2.5A). The main excavation was also inspected by a geologist from the NRC (see FSAR Attachment 2.5B). The inspection and mapping of the excavation confirmed the data obtained from the test borings taken in the site area (Figure 2.5-17). The excavation mapping program is discussed in Subsection 2.5.4.3.1.1.

2.5.1.2.2 Physiographic Setting

The site is located within a physiographic division of Illinois named the Kankakee Plain (Reference 47) as shown on Figure 2.5-4. The Kankakee Plain occupies a relatively small portion of a larger physiographic division called the Till Plains Section of the Central Lowland Physiographic Province (Reference 1).

The Till Plains Section is characterized by widespread and variable deposits of glacial till, outwash, and lacustrine sediments assigned primarily to the Wisconsinan and Illinoian glacial stages. The pre-glacial bedrock surface is irregular. A number of completely and partially buried bedrock valleys exist throughout the section. Overburden thickness is locally dependent on a combination of bedrock conditions and the amount of post-glacial erosion.

The Kankakee Plain is characterized by relatively low-lying and flat topography, except where it has been deeply incised by such major streams as the Illinois, Kankakee, Des Plaines, Du Page, and Mazon Rivers and their tributaries. This distinct physiographic division originated during a time when runoff of glacial melt-water flooded low areas between arch-shaped moraines and formed extensive glacial lakes such as Lakes Wauponsee and Watseka, as shown on Figure 2.5-4. Deposits of sand and gravel accumulated in this lacustrine environment. When the waters subsided, the major rivers became entrenched, terraces were formed, and large areas of lacustrine sand were exposed to wind action. Dunes were formed by prevailing westerly winds in some areas. They are not evident within the site area; however, strong cross-bedding suggestive of wind action is characteristic of the near-surface sand deposits throughout the site. Gradients of the natural ground surface within the site area are generally less than 1%. A physiographic map of the site area is shown on Figure 2.5-18.

2.5.1.2.3 Geologic History

2.5.1.2.3.1 General

A study of geology reveals the earth to be in a constant state of change. The lowest rocks in the stratigraphic sequence contain the oldest records of geologic history, and successively higher rocks provide an orderly record of the changing conditions at any given geographic location. The stratigraphic columns shown on Figures 2.5-2 and 2.5-19 provide a graphic illustration of the earth's changing history. The geologic history of the site area is derived partly from exposures of soils and rock and partly from nearby wells and borings. In addition, these data are supplemented by knowledge derived from adjacent regions. A surficial geologic map of the site area is shown on Figure 2.5-20. The ages for the geologic periods discussed below are taken from Reference 2.

2.5.1.2.3.2 Precambrian (Greater Than Approximately 600 Million Years B.P.)

In the site area, Precambrian rocks of undetermined composition lie at a depth of 4400 to 4500 feet below sea level. Since no borings at or near the site have reached Precambrian rocks, their nature and history must be inferred from observations in other areas. Data suggest that it is largely a granitic surface, with occasional dikes and patches of volcanic flow rock. The surface appears to be dipping southerly at approximately 60 ft/mi. Precambrian events discussed in Subsection 2.5.1.1.1.2 are assumed also to have occurred within the site area.

2.5.1.2.3.3 Cambrian (Approximately 500 to Approximately 600 Million Years B.P.)

No onsite boring has penetrated Cambrian-aged deposits; however, limited data are available from onsite water wells and deep borings located 2 to 10 miles from the site. Indications are that the site area was submerged during late Cambrian time. The first deposits in the advancing sea were coarse sand and fine pebbles, followed by finer sand, dolomite, and shale, with an increasing amount of calcareous material. Before the close of Cambrian time, the seas cleared, and chemical and/or organic precipitates which formed dolomite were deposited.

At the close of Cambrian time, the site area was uplifted. Minor advances and retreats of the sea deposited alternating layers of clastic and calcareous sediments, followed by a brief period of erosion.

2.5.1.2.3.4 Ordovician (430 ± 10 to Approximately 500 Million Years B.P.)

The Ordovician period began with a readvance of the sea. Initially, the deposition was predominantly medium-grained sand with minor calcareous zones. Later, general conditions favored the accumulation of calcareous deposits. Finally, there were alternating periods of accumulation of sand and calcareous materials.

Eventually the sea receded and a prolonged period of erosion was initiated. It appears, however, that the site area was not eroded as extensively as in adjacent localities.

Following the period of widespread erosion, the sea advanced over the existing erosional site topography, reworking the unconsolidated soils, then depositing a considerable quantity of fine to medium sand. Gradually the sea became clearer, and a thick sequence of calcareous sediments was deposited.

Onsite boring data indicate that a brief period of emergence followed the calcareous deposition. The site area was again submerged by the sea, and early conditions favored the deposition of silt and clay with minor amounts of calcareous material. Later the seas again became clear, and calcareous deposits dominated. Finally, conditions again favored the accumulation of silt and clay. The Ordovician period ended by elevation of the region above sea level followed by long, continued erosion.

2.5.1.2.3.5 Silurian (400 ± 10 to 430 ± 10 Million Years B.P.)

Silurian-aged deposits are rare at the site; however, regional geology indicates that they completely blanketed the site at one time. Data from adjacent areas reveal that, following the period of erosion at the end of Ordovician time, Silurian seas deposited a sequence that began with clastic sediments and ended with a thick carbonate accumulation. This period of deposition was followed by uplift and then erosion which probably removed most of the Silurian deposits in the site area except for an occasional outlier.

2.5.1.2.3.6 Devonian (340 ± 10 to 400 ± 10 Million Years B.P.)

Devonian sediments are not present within the site area; however, some marine deposition is believed to have taken place during late Devonian time. The character of these deposits is not definitely known. However, regional data suggest that they may have been primarily silty in nature. Uplift of the site area terminated the Devonian deposition. Subsequent erosion removed all traces of the Devonian deposits and perhaps additional Silurian rocks as well.

2.5.1.2.3.7 Mississippian (320 ± 10 to 340 ± 10 Million Years B.P.)

No Mississippian-aged deposits are present at the site. It is possible that a period of erosion, initiated during the close of Devonian time, persisted through Mississippian time. Erosion did penetrate well into the Ordovician deposits throughout most of the site area, as indicated by the pre-Pennsylvanian unconformable surface.

2.5.1.2.3.8 Pennsylvanian (270 ± 5 to 320 ± 10 Million Years B.P.)

During Pennsylvanian time at the site, sediments initially accumulated in a terrestrial environment. Lithologies indicate the occurrence of periods of rapid accumulations of alluvial silt and very slow accumulations of clay under probably stagnant conditions. Abundant vegetation flourished and accumulated in significant quantities. Many of these changes in environment were locally separated by periods of nondeposition or erosion.

After the period during which vast stretches of vegetation flourished and accumulated, the environment changed. A shallow sea encroached on the site area. Near-shore deltaic deposits of fine-grained materials accumulated as streams emptied into the sea. Limestone was deposited locally. As the delta prograded, laminated siltstones were deposited in distributary and interdistributary bays (Reference 48). Distributary stream channels cut into the existing delta and deposited beds of silty sand. Some of these deposits may have been reworked to form delta front sands. Cross-cutting of channels formed complex arrangements of bedding and lithology in some places.

It is not definitely known if sediments younger than the Pennsylvanian were ever deposited in the site area until Pleistocene time.

2.5.1.2.3.9 Quaternary (Present to 2 ± 1 Million Years B.P.)

Beginning with Pleistocene time, the site area is believed to have been invaded by a number of glacial advances; however, only deposits from the midpart of the Woodfordian substage of the Wisconsinan glacial advance are present within the site area. It is likely that deposits of previous glacial advances were removed by subsequent advances. Radiocarbon dates place the basal Woodfordian at approximately 22,000 radiocarbon years B.P. The probable radiocarbon age for the top is about 12,500 B.P. (Reference 6). Radiocarbon dates for the middle Woodfordian are few.

The Valparaiso glacier attained its maximum extent during the Woodfordian Substage of the Wisconsinan Stage and deposited the Valparaiso moraine as shown on Figure 2.5-6. As the glacier receded, meltwater from a vast area escaped through the Des Plaines, Du Page, Kankakee, Fox, and Illinois valleys. The abundance of Valparaiso outwash along hundreds of miles of icefront is evidence that unusually large volumes of meltwater issued from the ice. In the Kankakee valley, the water constituted a torrent which transported and deposited slabs of limestone. The volume of the Kankakee torrent supplemented by glacial water from other valleys was so great that it could not escape along the Illinois valley and consequently formed several glacial lakes which inundated low-lying areas between various glacial moraines. The site area was occupied by glacial Lake Wauponsee, which rose temporarily to an elevation of about 650 feet above sea level. The major currents eroded a wide gap in the Minooka moraine and carried a large quantity of sand and silt into Lake Wauponsee. The lake plain south of Coal City is generally underlain by sand. To the north the deposits are essentially silt with some thin beds of sand and clay.

Following the Kankakee torrent and the lowering of glacial Lake Wauponsee, the lacustrine silts and sands were exposed to erosion. Wind-blown sands were deposited in the site area in sheet-like deposits and occasional dunes. After most of these wind-blown deposits were stabilized by vegetation, modern soils slowly developed.

2.5.1.2.4 Stratigraphy

The stratigraphy at the Braidwood site is well known, both from published material and from numerous borings completed during the course of this investigation. The excavation mapping program confirmed the interpretation of the subsurface geology discussed in the Braidwood PSAR. During the excavation mapping program it was found that the Parkland Sand could be differentiated from the Equality Formation (see Subsections 2.5.1.2.4.1.1.1 and 2.5.4.3.1.1). While the ability to differentiate these units has increased knowledge of the site geology, it has no effect on the design or construction of the Braidwood Station.

In general, the site is underlain by a regular sequence of units marked by a remarkable uniformity and continuity. Units such as the Colchester (No. 2 Coal) member horizon of Pennsylvanian age are extremely persistent and serve as excellent marker beds. Detailed examination of borings and other subsurface data indicate a lack of faulting in the Colchester (No. 2 Coal) member and provide strong support for the concept that faulting is not a significant structural element at the Braidwood site. A detailed discussion of the stratigraphy follows.

2.5.1.2.4.1 Soil Deposits

Overburden deposits within the plant site area consist of eolian deposits, lacustrine deposits, outwash, and glacial till. Borings at the site vicinity encountered soil deposits which ranged in thickness from 26.0 feet in Boring MP-46 to 62.0 feet in Boring P-10. The average soil thickness encountered in the site borings was approximately 42.0 feet. The sequence and nature of the soil and rock units within the site area are shown in a composite stratigraphic column, Figure 2.5-19, in a fence diagram of the site area, Figure 2.5-21, and in subsurface cross sections, Figures 2.5-22 through 2.5-26.

2.5.1.2.4.1.1 Pleistocene

Deposits of Pleistocene age within the site area consist of soils which are associated either directly or indirectly with Pleistocene glaciation. They can be divided into upper and lower units on the basis of origin and distinct sedimentary characteristics. These have been classified by Pleistocene stratigraphers as the Parkland, Equality, and Wedron Formations.

2.5.1.2.4.1.1.1 Parkland Sand

The Parkland Sand consists of wind-blown sand which blankets the site vicinity in sheet-like deposits except in strip-mined areas and areas where wind erosion has exposed the underlying Equality Formation (blowouts). Some small stabilized dunes of Parkland Sand are also found in the site vicinity.

At the site, the Parkland Sand is typically a light brown to reddish-brown, silty, very fine to fine sand which has been derived from the underlying silts and sands of the Equality Formation.

This formation was probably present in many of the site borings and can be distinguished in the boring logs by the silty, light brown to reddish-brown fine sand in the upper few feet of the Equality Formation.

The Parkland Sand was determined to be present at the site as a separate unit from the Equality Formation during the excavation mapping program. This interpretation was later confirmed by members of the Illinois State Geological Survey (see FSAR Attachment 2.5A).

In geologic sections of the main plant excavation, the Parkland Sand ranged in thickness from 5.7 to 9.5 feet, with an average of 7.4 feet in those areas where it was undisturbed.

The Parkland Sand and the Equality Formation are not differentiated on the boring logs and the test pit logs (Figures 2.5-123 through 2.5-260) and the geologic cross sections derived from these logs (Figures 2.5-22 through 2.5-26). The Parkland Sand is shown separately from the Equality Formation on the site stratigraphic column (Figure 2.5-19) and the geologic sections made during excavation mapping (see Subsection 2.5.4.3.1.1 and Figure 2.5-50).

2.5.1.2.4.1.1.2 Equality Formation

The Equality Formation consists of lacustrine sands and silts and is subdivided into the Dolton and Carmi Members (Reference 6). Only the Dolton Member is present at the site, and it blankets the entire site vicinity except where it has subsequently been strip-mined.

The Dolton Member within the site area consists primarily of yellowish-brown to gray, fine sand which was deposited primarily in beaches and bars of glacial Lake Wauponsee as discussed in Subsection 2.5.1.2.3.9 (Reference 6).

The Dolton lacustrine sand was penetrated by 114 borings throughout the site area. The upper few feet of the deposit is a fine sand which typically is oxidized to a yellowish-brown color, generally contains less than 15% silt, and has a consistency ranging from loose to medium dense. Below a depth of 15 feet, the sand grades from brown to a grayish-brown and a light gray color, generally contains less than 5% silt, and has a consistency ranging from medium dense to dense. Lag gravels are found locally at the base of this sand. The overall consistency of the Dolton Member is medium dense. Ground surface elevations at the site for areas not strip-mined range from 591.7 to 603.6 feet. The average elevation at the main plant site is 600.4

feet. Based on the results of borings drilled at the site, the Dolton Member ranges in thickness from approximately 14.0 feet to 31.2 feet and averages approximately 23.0 feet.

The lacustrine sand is permeable and water-bearing. The groundwater, which originates chiefly from surface infiltration, is perched on the underlying glacial till deposits of low permeability.

An examination of lacustrine sand exposures along a vertical strip-mining cut adjacent to the plant site revealed strong cross-bedding. Most of the groundwater has drained naturally from the exposed permeable sand. The basal 3 to 4 feet continue to dewater, forming a continuous line of seepage along the top of the underlying glacial till deposits. Dewatering sand exposures with slopes which approach 70° to 80° from horizontal were observed.

2.5.1.2.4.1.1.3 Wedron Formation

The Wedron Formation appears to underlie the entire site wherever the Dolton lacustrine sand was encountered. Based on field identification and laboratory analysis by the Illinois State Geological Survey, two members of the Wedron Formation have been identified in the main plant area. These are the Yorkville and Tiskilwa Till Members.

Underlying the Tiskilwa Till Member are several feet of glacial deposits which may be as old as the Wedron Formation or which may be older (see FSAR Attachment 2.5A).

For ease of discussion, the Yorkville and Tiskilwa Till Members and the underlying glacial deposits will be referred to as the Wedron Formation.

The Wedron Formation was penetrated by 111 borings throughout the site area. It frequently consists of three units: an upper till consisting predominantly of dark gray clayey silt to silty clay with interspersed sand and dolomitic gravels, underlain by an outwash layer of grayish-brown sandy gravel to gravelly sand with numerous cobbles and some boulders, and a lower till consisting predominantly of a brownish-gray to gray, very sandy silt with some interspersed clay and gravel. These three basic units, however, are extremely variable in thickness, and any combination of them, in their proper sequence, constitutes the Wedron Formation within the site area.

As a whole, the Wedron Formation was observed in onsite borings to vary in thickness from 4.5 feet in Boring MP-6 to 29.5 feet in Boring MP-57; it has an average thickness of 17.8 feet. The top of the formation lies between elevations of 568.9 feet and 583.7 feet, with an average elevation of 576.4 feet. Figure 2.5-29 shows the contoured surface of the Wedron Formation for the site area and the main plant area.

In addition to the three units within the Wedron Formation, minor amounts of reworked soils were encountered along the lower formation contacts. For the purpose of this report, these reworked soils are considered to be part of the Wedron Formation. These soils appear to be somewhat sporadic in occurrence and may be encountered at any location within the site area. The reworked soils are composed of gray silt with a trace of clay and gray, sandy, and micaceous silt with some sandstone gravels.

Outwash exposures along the strip-mine cut were water-bearing and formed a continuous line of seepage with conspicuous iron stains. Cuts through the till and outwash were at vertical slopes.

2.5.1.2.4.2 Bedrock Deposits

The bedrock deposits in the vicinity of the site range in age from Pennsylvanian to Precambrian as shown in Figures 2.5-2 and 2.5-19. The elevation of the bedrock surface ranges from 551.9 feet to 567.0 feet and averages 558.3 feet. Figures 2.5-30 and 2.5-31 are contour maps of the bedrock surface which is formed in the upper Pennsylvanian deposits.

The deepest borings drilled at the site were extended into the upper portion of the Ordovician-age Galena Group. The site water well penetrated into the Cambrian-age Ironton and Galesville Sandstones. Descriptions and estimated stratigraphic thicknesses of deeper units were obtained from the following sources:

a. a deep well in the town of Braidwood, Illinois, illustrated in Figure 2.5-32 (Reference 49);

b. a composite of wells in Sections 28, 29, and 32 in T.30N., R.10E. (Reference 13);

c. the log of a well in Section 25 in T.34N., R.9E. (Reference 13); and

d. the log of the site water well in Section 19, T.32N., R.9E. (see Figure 2.5-16, Sheet 3, and FSAR Attachment 2.5C).

Rock quality designation (RQD) has been used as an indication of the general quality of the rock. This procedure employs a modified core recovery percentage in which only the pieces of sound core 4 inches or longer are counted as recovery. The cumulative length of pieces 4 inches or longer is then expressed as a percentage of the total length of the core run.

The core from the initial 30 borings (the A, P, L, and H series) was obtained using NX and HQ-wireline double-tube core barrels. The main plant location borings (the MP series) used mainly NX double-tube core barrel for rock coring. RQD is a function of drilling techniques. Therefore, as the largest number of borings were drilled using the NX double-tube core barrel, only the borings drilled by this method in the main plant area will be used to compare rock quality and recovery.

2.5.1.2.4.2.1 Pennsylvanian

Deposits of Pennsylvanian age which underlie the site consist of bedrock. Some residual soil was encountered in several borings, although none was found at the main plant site. The soil was formed as a result of inplace weathering of Pennsylvanian bedrock and ranges somewhat in character from gray to bluish-gray silt with various amounts of clay and fine sand, to light gray, silty, fine or fine-to-medium sand. Mica flakes are conspicuous throughout. Although bedding generally is not evident in the silty sand layers, the silts are without exception thinly laminated. The residual soil, an inplace weathering product of the underlying Carbondale Formation bedrock, rests in most places on a siltstone of the Francis Creek Shale Member of the Carbondale Formation. It was, however, locally encountered on the channel sandstone or limestone which overlie the siltstone in the Francis Creek Shale Member. Where encountered in the borings, the residual soil has a hard consistency.

All the Pennsylvanian bedrock is included within the Kewanee Group, which is subdivided into the Carbondale and Spoon Formations. The Pennsylvanian deposition in the site area is characterized by rapid vertical changes in rock type and by lateral persistence of the Colchester (No. 2) Coal Member of the Carbondale Formation. Sandstone, siltstone, and most shale units are also persistent over wide areas when viewed as composite units. However, they show noticeable variation in thickness over relatively short horizontal distances.

2.5.1.2.4.2.1.1 Kewanee Group

2.5.1.2.4.2.1.1.1 Carbondale Formation

2.5.1.2.4.2.1.1.1.1 Limestone

A light gray to buff, very silty limestone was encountered near the top of bedrock in two borings. It is thinly bedded, highly fractured, and moderately weathered. No evidence of solution activity was observed. Thicknesses varied from 2.0 feet in Boring A-10 to 5.0 feet in Boring A-7.

Where limestone was encountered, the RQD ranges from 0% to 8%, and the percentage of core recovery ranged from 60% to 90%. The low values of RQD may be attributed to weathering of the upper bedrock layers.

2.5.1.2.4.2.1.1.1.2 Francis Creek Shale Member

2.5.1.2.4.2.1.1.1.2.1 Channel Sandstone Deposits

A thin to locally thick sandstone unit was encountered at or near the top of bedrock at many locations, particularly in the main plant area. Stratigraphically it lies below the locally present silty limestone and above the siltstone of the Francis Creek Shale Member. The sandstone unit has been classified by the Illinois State Geological Survey as part of the Francis Creek Shale Member on the basis of both spore analyses and on the coal within the sandstone (Reference 50). The channel deposits consist of light gray, silty, fine- or fine-to-medium-grained sandstone with occasional interbedded shale layers. The unit as a whole is micaceous and thin-bedded. Conglomeratic sandstone, with clasts (angular fragments) and stringers of coal and pebbles (probably concretions) of siderite, is locally well developed at or near the base. The contact with the underlying siltstone is disconformable.

Where the sandstone is present, thicknesses vary from 3.0 feet in Boring A-9 to 27.3 feet in Boring A-3. The average thickness from a total of 62 borings is 7.4 feet.

The RQD ranged from 0% to 100% and averaged 48.1%. The percentage of core recovery averaged 82.2% and ranged from 8% to 100%. Low values of both RQD and recovery are due to various degrees of weathering which have taken place along the upper zones of bedrock. In many cases, due to poor cementation or lack of cementation of the sandstone, the initial core was washed away in the drilling process.

2.5.1.2.4.2.1.1.1.2.2 Siltstone Deposits

The siltstone of the Francis Creek Shale Member is present throughout the site area except where it has been removed by strip-mining the underlying No. 2 Coal. It represents a wedge of clastic sediments from a northern or eastern source which was deposited rapidly soon after the No. 2 Coal deposition. It can essentially be described as a gray, micaceous siltstone which grades from sandy at the top to a finely micaceous, silty shale near the base. It is thinly laminated throughout. Siderite concretions are common but are only occasionally fossiliferous.

Within the siltstone is a zone of conglomeratic sandstone. This zone is marked by large clasts and occasional beds of coal and pebbles of siderite. The matrix is frequently calcareous, though in most places it is a fine- to medium-grained sandstone. The top of the conglomeratic zones ranges in elevation from 511.6 to 553.3 feet and ranges in thickness from 1.2 feet in Boring MP-47 to 36.9 feet in Boring A-14.

The overall thickness of the siltstone deposits averages 51.8 feet. The thickness ranges from 28.5 to 65.5 feet. RQD values for the siltstone ranged from 0% to 100% and averaged 77.1%. The percentage of core recovery averaged 96.3% and ranged from 33% to 100%. RQD values in the plant area are varied due to irregular amounts of weathering in the upper portions and the more fissile nature of the rock as it grades to shale near the base.

Laboratory test results indicate unconfined compressive strengths ranging from 2420 to 7286 psi for seven of eight samples. One low strength of 980 psi was due to a concealed shear break in the sample (see Table 2.5-3). Results of resonant column tests performed on four samples from the siltstone deposts are given in Table 2.5-4.

Visual observations were made of the exposed siltstone deposits along vertical strip-mine cuts adjacent to the site. Differential weathering of the thinly laminated silty shale (less resistant) and siltstone (more resistant) layers had developed an irregular surface with numerous horizontally oriented knife edge projections. In addition, numerous irregular short fractures had developed across bedding planes. In general, weathering had not penetrated more than 3 or 4 inches. The exposures were, on the whole, very stable along vertical cuts and did not display the slaking tendency so common to shales with a high clay content.

2.5.1.2.4.2.1.1.1.3 Colchester (No. 2 Coal) Member

The Colchester, one of the most continuous beds in the Pennsylvanian of Illinois, consists of thinly bedded to thinly laminated layers of dull to bright black coal but contains no persistent partings. Numerous vertical fractures are present which contain thin veinlets of pyrite and clay. Elevations at the top of the coal ranged from 486.1 to 518.3 feet averaging 501.0 feet. Figure 2.5-33 is a contour map at the main plant site of the top of the Colchester Member and indicates the thickness of coal present at each boring location. Stratigraphically, the base of the No. 2 Coal forms the base of the Carbondale Formation.

Observed thicknessses vary from 1.5 feet in Boring MP-25 to 4.7 feet in Boring A-5. The average thickness from a total of 83 borings is 3.3 feet.

RQD averaged 53.6% and ranged from 0% to 100%. The percentage of core recovery averaged 91.4% and ranged from 59% to 100%. Low RQD values for the coal are attributed to breakage during drilling and probably do not represent true in situ conditions.

2.5.1.2.4.2.1.1.2 Spoon Formation

The Spoon Formation is the lowest recognized Pennsylvanian unit which underlies the site. Its basal contact with the underlying Ordovician deposits is locally sharp and easily recognized. Several distinct sedimentary units have been distinguished within the Spoon Formation as observed in onsite borings. These consist essentially of interbedded clayey shales, silty shales with carbonaceous zones, and siltstones or silty sandstones. Abrupt vertical changes in lithology and rapid lateral variations in thickness are characteristics of these units.

The clayey shale units within the Spoon Formation are generally carbonaceous and brownish-gray to dark gray in color. The shales are massive and highly fragmented in nature. The fractures are small, extremely numerous, irregular, slickensided, and have no definite pattern of orientation. The fractures are believed to be related to desiccation and compaction subsequent to burial rather than to tectonic activity. Siltstone layers both above and below the fragmented units are rarely fractured or slickensided.

The silty shale intervals are generally dark gray to black in color. They grade carbonaceous in zones with well-developed coal layers in some locations. A maximum of 6.8 feet of coal was observed in Boring A-5 below the No. 2 coal layer. The silty shale layers are thinly laminated to massive. Zones with randomly oriented fragmentation and slickensides are typical. The fragmentation and formation of slickensides in the silty shale is also believed to be the result of desiccation and compaction subsequent to burial rather than the result of tectonic activity.

Three light gray and greenish-gray sandy siltstone and silty sandstone beds are present in the main plant area. These layers are micaceous, slightly carbonaceous, and contain sand-size grains of siderite throughout. Bedding is thin but grades indistinct in zones give a deceptively massive appearance to the fresh cores. Fractures are rare. The thickness of the three layers represents about 6 to 10 feet of the total Spoon Formation at the site.

The lower portion of the Spoon consists of dark gray, sandy, carbonaceous siltstone. The siltstone is highly fissile near the base and parts on laminae of carbonized plant fossils.

The Spoon Formation as a whole was observed to range in thickness from 11.7 feet in Boring A-5 to 48.5 feet in Boring A-15. An average thickness from a total of 57 borings is 33.8 feet.

Core recovery for the Spoon Formation ranged from 31% to 100%, averaging 93%. RQD averaged 67% and ranged from 5% to 100%. The shales in this formation tend to swell in the core barrels. Breakage occurs when the rock is extracted from the core barrels, and RQD values are undoubtedly lower than in situ conditions. Although the gray to greenish-gray siltstone has relatively high RQD values, the dark gray basal siltstone may exhibit frequent bedding plane parting, yielding lower RQD values.

Laboratory testing could not be performed on the fragmented shales, but the light gray and greenish-gray siltstone units were utilized. Unconfined compressive strengths ranged from 4020 to 8600 psi for five samples of siltstone tested (see Table 2.5-3). Other rock types within this formation are anticipated to have much lower unconfined strengths.

2.5.1.2.4.2.2 Silurian

2.5.1.2.4.2.2.1 Alexandrian Series

Silurian deposits were not anticipated to underlie the site area; the general outcrop edge is mapped a few miles to the east (Figure 2.5-5). However, it appears that small scattered patches of Silurian sediments exist as outliers west of the main body. That these patches were preserved in pre-Pennsylvanian topographic lows is suggested by a thin layer of reworked Ordovician shale which overlies the Silurian in Boring L-4. The reworked shale appears to have been transported from Ordovician exposures at some higher elevation and deposited during initial stages of the Pennsylvanian period. Regional unconformities occur at the upper and lower contacts of Silurian rocks.

Silurian deposits were encountered in two borings. Total thickness varied from 31 feet in Boring L-3 to 33 feet in Boring L-4. Deposits consisted of 17 to 25 feet of light brown to greenish-gray dolomite. It is fine- to medium-grained, silty, and thinly bedded, with numerous thin irregular green shale partings. The dolomite is underlain by 7.5 to 14.0 feet of light greenish-gray to dark gray dolomitic siltstone which is thinly laminated.

Fractures are generally tight and widely spaced. The RQD ranges from 83% to 93%. The percent of core recovery ranges from 98% to 100%. Porous zones are limited to small vugs (1-inch maximum) and pinpoint openings.

2.5.1.2.4.2.3 Ordovician

2.5.1.2.4.2.3.1 Maquoketa Shale Group

2.5.1.2.4.2.3.1.1 Brainard Shale

The Brainard Shale, where present within the site area, forms the top of Ordovician deposits and rests conformably on the Fort Atkinson Limestone. At some locations, the Brainard is very thin to absent due to the erosional unconformity between the Pennsylvanian and Ordovician sediments. The formation consists of greenish-gray silty and dolomitic shale with occasional marine fossils. Bedding is thinly laminated. The shale is generally well indurated with few fractures.

The Brainard Shale is variable in thickness over short horizontal distances. It was observed to vary from a few inches in Boring A-8 to 78.5 feet in Boring L-2. An average thickness from a total of 21 borings is 14.1 feet. The Brainard Shale was not encountered in any of the main plant borings.

The RQD ranges from 46% to 98%. The percent of core recovery ranges from 92% to 100%. These figures are taken from borings drilled with various drilling techniques, but due to the small number of borings where Brainard was encountered, a single drilling technique was not isolated for evaluation of RQD and recovery.

Laboratory test results indicate an unconfined compressive strength of 7142 psi from one sample taken from the basal calcareous zone. Strengths of the upper Brainard are anticipated to be somewhat less, but suitable samples are difficult to obtain because of the tendency of the upper Brainard to part along bedding planes upon exposure to the atmosphere.

2.5.1.2.4.2.3.1.2 Fort Atkinson Limestone

This formation is present throughout the site area. Where the Brainard has been removed, the upper contact of the Fort Atkinson is unconformable with Pennsylvanian deposits. The elevations of the Fort Atkinson range from 434.2 to 475.1 feet and average 451.0 feet (see Figure 2.5-34). The contact with the underlying Scales Shale appears to be conformable. The Fort Atkinson consists essentially of gray, silty limestone which grades from light gray to buff downward. The silty limestone is thinly bedded and contains numerous irregular partings and gradational zones of dark gray, calcareous shale. Numerous fossils occur throughout. The light gray to buff limestone grades coarsely calcarenitic and free of silt. It is thin- to medium-bedded with occasional thin shale partings and stylolites. Minor oil-stained vugs and porous zones occur in basal zones. The pores are generally very small, rarely larger than 1 inch in diameter.

Observed thicknesses of the Fort Atkinson Limestone range from 29.9 feet in Boring L-3 to 41.4 feet in Boring A-15. The average thickness from a total of 28 borings is 38.2 feet.

The RQD ranged from 21% to 100% and averaged 92.3%. The percent of core recovery ranged from 50% to 100% and averaged 98.4%.

Laboratory test results indicate unconfined compressive strengths ranging from 3,121 to 14,333 psi for eight of nine samples (see Table 2.5-3). One low strength of 1878 psi was due to a concealed shear break in the sample. Results of resonant column tests performed on four samples from the Fort Atkinson Limestone are given in Table 2.5-4.

2.5.1.2.4.2.3.1.3 Scales Shale

The Scales Shale is the basal formation within the Maquoketa Shale Group. It rests unconformably on sediments assigned to the Galena Group and is present throughout the site area. The upper

6.5 to 19.0 feet is a light gray, calcareous siltstone which is extremely fossiliferous. This grades downward to gray, silty, calcareous shale with interbedded zones of silty limestone, and finally to dark gray, dolomitic shale with occasional thin layers of calcareous siltstone. In general, bedding is thickly laminated and somewhat irregular. Upon exposure, the shale parts readily along bedding planes. Fractures are rare.

Observed thicknesses range from 86.2 feet in Boring A-2 to 90.9 feet in Boring L-2. The average thickness from eight borings is 88.4 feet.

The RQD varies from 96% to 100% and averages 98.3%. The percent of core recovery ranges from 96% to 100% and averages 98.5%.

Laboratory test results indicate unconfined compressive strengths of 5918, 6660, and 8469 psi (see Table 2.5-3).

2.5.1.2.4.2.3.2 Galena Group

2.5.1.2.4.2.3.2.1 Wise Lake and Dunleith Formations

The Wise Lake and Dunleith Formations in northeastern Illinois are so similar that they cannot be readily separated in boring samples. This combined unit is the lowest stratigraphic horizon reached by onsite borings. A maximum of 27 feet was cored in Boring L-2; however, the unit was not entirely penetrated. The upper contact with the overlying Scales Shale is easily distinguished even though an unconformity is indicated. Irregular small voids in the top of the Wise Lake-Dunleith are filled with fossiliferous, silty limestone which grade upward into the basal Scales Shale. The cored portions of the Wise Lake-Dunleith Formations consist of mottled, light gray to buff, fine- to medium-crystalline, dolomitic limestone. Bedding is thin with numerous very thin and irregular shale partings. Fractures are not common. Vugs and localized porous zones are present at some localities and entirely absent in others. Solution activity is not a significant factor in this unit.

Total thickness within the site area is estimated to be between 165 and 245 feet on the basis of deep boring data in adjacent areas. A structural contour map on top of the Galena is presented in Figure 2.5-35.

The RQD varies from 97% to 100%. The percent of core recovery varies from 99% to 100%.

Laboratory test results from one sample indicate an unconfined compressive strength of 9591 psi.

2.5.1.2.4.2.3.2.2 Guttenburq Formation

The Guttenburg Formation is a light buff to grayish-brown, fine to medium-crystalline, medium- to massive-bedded dolomite with thin, typically reddish-brown shale partings. Thickness in the site area is estimated to be 10 to 20 feet on the basis of deep boring data in adjacent areas.

2.5.1.2.4.2.3.3 Platteville Group

2.5.1.2.4.2.3.3.1 Nachusa Formation

The formation is a buff to grayish-brown, fine- to medium crystalline, medium- to massive-bedded dolomite with thin gray shale partings and occasional chert nodules. Thickness in the site area is estimated to be 34 to 48 feet on the basis of deep boring data.

2.5.1.2.4.2.3.3.2 Grand Detour Formation

The Grand Detour Formation is a brownish-gray, finely crystalline, fossiliferous, medium- to massive-bedded dolomite with gray and reddish-brown shale partings. This formation commonly has gray mottling and small amounts of chert nodules. Thickness on the site area is estimated to range between 21 and 40 feet on the basis of deep boring data.

2.5.1.2.4.2.3.3.3 Mifflin Formation

The Mifflin Formation is a finely crystalline, thin- to medium-bedded, light gray or buff dolomite and limestone with bluish-gray, gray, green, or brown shale partings, rare chert nodules, and zones of orange speckling. Thickness in the site area is estimated to range between 35 and 50 feet on the basis of deep boring data.

2.5.1.2.4.2.3.3.4 Pecatonica Formation

The Pecatonica Formation is light grayish brown, finely crystalline dolomite, with medium to massive bedding. This formation contains thin brown shale partings and may contain some brown to brownish-gray dolomite and dolomitic limestones. Thickness in the site area is estimated to be 34 to 48 feet on the basis of deep boring data.

2.5.1.2.4.2.3.4 Ancell Group

2.5.1.2.4.2.3.4.1 Glenwood Formation and St. Peter Sandstone

The Glenwood Formation and St. Peter Sandstone are not differentiated in the site area. In general, the Glenwood is a fine- to coarse-grained dolomitic sandstone with some light green shale, while the St. Peter is a fine- to medium-grained quartzose sandstone, poorly cemented and friable. Total thickness of these combined formations in the site area is estimated to be between 157 to 540 feet on the basis of deep boring data.

2.5.1.2.4.2.3.5 Prairie du Chien Group

2.5.1.2.4.2.3.5.1 Shakopee Dolomite

The Shakopee Dolomite is primarily a very finely crystalline, light gray to light brown dolomite. It contains oolitic chert and some light gray to green shale. It may contain lenses of massive algal structures up to 10 feet high. Where present in the site area, it may range up to about 67 feet in thickness.

2.5.1.2.4.2.3.5.2 New Richmond Sandstone

This formation is a buff, moderately sorted, rounded, friable, medium-grained sandstone with some interbedded, light-colored, sandy dolomite. Where present in the site area, it may range up to 10 feet in thickness.

2.5.1.2.4.2.3.5.3 Oneota Dolomite

The Oneota Dolomite is estimated to be from 8 to 250 feet thick within the site area. It is divided into two members chiefly on the basis of chert content. The upper or Blodgett Member consists of noncherty to slightly cherty dolomite. The lower Arsenal Member consists of cherty to very cherty dolomite. The dolomite in both units is light gray to pink, coarse-grained, and has minor amounts of sand.

2.5.1.2.4.2.3.5.4 Gunter Sandstone

The Gunter Sandstone consists of medium-grained, friable, subrounded sandstone that contains beds of light gray, fine-grained dolomite and minor amounts of green shale. Where present in the site area, it may range up to about 7 feet in thickness.

2.5.1.2.4.2.4 Cambrian

2.5.1.2.4.2.4.1 Eminence Formation

The Eminence Formation is a light gray to light brown, sandy, fine- to medium-grained dolomite with some oolitic chert and thin beds of sandstone. The thickness within the site area is thought to be about 82 to 93 feet on the basis of deep boring data. The name Momence Sandstone Member has been proposed (Reference 13) for the 5 to 15 foot-thick discontinuous sandstone, which is sometimes found at the base of the Eminence Formation.

2.5.1.2.4.2.4.2 Potosi Dolomite

This formation is a finely crystalline, slightly argillaceous, brown to light gray dolomite. It is generally slightly glauconitic near the top and glauconitic and sandy near its base. Its thickness is estimated to be between 162 and 212 feet within the site area.

2.5.1.2.4.2.4.3 Franconia Formation

This formation consists of a light gray to pink, fine-grained, dolomitic sandstone that is usually glauconitic, silty, and argillaceous. Its thickness is estimated to be between 88 and 142 feet within the site area.

2.5.1.2.4.2.4.4 Ironton Sandstone

The Ironton Sandstone is a medium- to coarse-grained, partly dolomitic, poorly sorted sandstone. The approximate thickness within the site area is estimated to be between 130 and 215 feet on the basis of deep boring data from adjacent areas. Four distinct members are recognized on the basis of lithology. From top to bottom, these are the Mooseheart, Marywood, Fox Valley, and Buelter Members.

The Mooseheart Member consists of poorly sorted dolomitic sandstone that is medium- to coarse-grained. Thickness within the site area is estimated to be between 45 and 60 feet.

The Marywood Member consists of fine-grained sandstone with little dolomite. Thickness within the site area is estimated to be between 5 and 50 feet.

The Fox Valley Member consists of poorly sorted, medium- to coarse-grained, dolomitic sandstone. Thickness within the site area is estimated to be between 20 and 25 feet.

The Buelter Member consists largely of medium-grained sandstone. It is moderately sorted and rarely dolomitic. Thickness within the site area is estimated to be between 60 and 80 feet.

2.5.1.2.4.2.4.5 Galesville Sandstone

The Galesville Sandstone is a white, fine-grained, friable sandstone which grades medium-grained and dolomitic in the basal portion. Thickness within the site area is estimated to be between 82 and 100 feet on the basis of deep boring data.

2.5.1.2.4.2.4.6 Eau Claire Formation

This formation consists of a variety of lithologies. In the site area, the primary lithologies are silty shale and siltstone which are dolomitic and glauconitic, underlain by approximately 80 feet of silty dolomite. The estimated thickness within the site area is approximately 562 feet on the basis of deep boring data.

2.5.1.2.4.2.4.7 Mt. Simon Sandstone

This formation is a fine- to coarse-grained, poorly sorted, friable sandstone which contains occasional fine pebbles. Coarse-grained beds are often cross-bedded. Red and green micaceous shales occur in beds from a few inches to 15 feet thick. The thickness within the site area is estimated to be somewhat in excess of 2460 feet on the basis of deep boring data.

2.5.1.2.4.2.5 Precambrian

No wells have reached the Precambrian in the site area. Available data indicate that the basement rocks consist largely of medium- to coarse-grained granite. Other rock types reported are quartz monzonite, rhyolite, porphyry, and felsite (Reference 51). Estimated depth to the top of the Precambrian is 4400 to 4500 feet below sea level (Reference 13).

2.5.1.2.5 Structure

2.5.1.2.5.1 Jointing

Joints are bedrock fractures along which no displacement has occurred parallel to the joint surface. They usually cut across bedding at a high angle but may vary from near horizontal to vertical. Lateral spacing between parallel fractures or joint sets varies considerably, and intersections of different joint sets are common.

Joints are present in the bedrock of the site area. A determination of their spacing and trend is not possible from surface observations due to a substantial thickness of unconsolidated deposits. Also, it is not possible to determine their trend or exact spacing from rock core samples. An examination of Pennsylvanian exposures above the No. 2 coal in the strip-mine area adjacent to the site indicates two distinct sets of joints. One set is spaced 1 inch to 16 inches, trends from due north to N 21° W, and dips from 80° to vertical. The second set is spaced 2 to 8 inches, trends N 73° E to N 99° E, and dips from 70° to 89°. All trends are from true north and dips are measured prependicular to the trends. All joints are tight, with no evidence of solution activity. During the excavation mapping, those joints observed were generally tight with no apparent movement along the joint surface.

2.5.1.2.5.2 Folding

Contour maps for the site area have been constructed on various stratigraphic horizons. These are illustrated on Figures 2.5-29 through 2.5-35. Contours on top of unconformable surfaces of erosion do not necessarily reflect site structure related to tectonic movement, as there is difficulty in determining the configuration of the erosional and/or depositional surface prior to deformation. Such unconformable surfaces are represented by the top of the Wedron Formation, by the top of bedrock and, to some extent, the tops of the Galena, Fort Atkinson, and Colchester units.

The period of erosion at the end of the Galena time is believed to have been brief. Since it is likely that erosion did not produce significant topographic relief on the Galena surface, present-day relief on this surface could conceivably be the result of tectonic movement which has taken place since Galena deposition.

The top of the Fort Atkinson Limestone is a comformable surface over portions of the site; however, pre-Pennsylvanian erosion cuts through the Brainard and into the Fort Atkinson in some locations.

Although structure contours on the top of the Fort Atkinson Limestone (Figure 2.5-34) indicate a local northeastward dip in the plant area, contours on the top of the Galena Group (Figure 2.5-35) in the broader site area closely conform to the southwestward dip of the site region (Figure 2.5-12).

Contours on top of the Galena indicate an anticlinal high which centers at or near Boring A-1 and appears to trend northwest - southeast across the site area. Sufficient data are not presently available to delineate this apparent structure in more detail. Regional data within the area suggest anticlinal structures which also trend in a northwest-southeast direction (Figure 2.5-12).

Irregular depositional and/or erosional surfaces within Pennsylvanian deposits at the site each display some relief independent of one another which should not be interpreted as due to tectonic movements. The surfaces do not reflect the structure observed within the Ordovician, such as the top of the Galena as previously discussed. Also, minor warping configuration shown on tops of both the Colchester No. 2 Coal Member and the Ft. Atkinson Limestone may be due as much to erosion as to tectonic activity and the fact that structures shown on one unit are similar to structures shown on another unit may be coincidental. However, it is possible that minor warping of the two geologic units may be due to tectonic forces acting on the LaSalle Anticlinal Belt. Clegg (1965, Page 93) indicates that structural and stratigraphic relationships in northern Illinois show that a second phase of deformation of the LaSalle Anticlinal Belt began after the deposition of the Colchester No. 2 Coal and possibly continued to the end of or after Pennsylvanian time.

Detailed correlations within the Pennsylvanian and younger deposits do not indicate any site movement over the past 200 million years. Major reported folds in the mid-continent are tabulated in Table 2.5-1 and are shown on Figure 2.5-8.

2.5.1.2.5.3 Faulting

Inspection of bedrock exposures in the strip-mining and the site excavations and detailed correlations of stratigraphic horizons penetrated by onsite borings have shown no evidence of faulting within the site area. The stratigraphic variations that do exist can be accounted for by local and regional unconformities.

The site is located in the tectonically stable interior region of the continent wherein faulting is not a major structural element. Extensive geologic work, both surface and subsurface, has been done; regional and local stratigraphy are well known; and, accurate stratigraphic correlations are possible throughout the area, owing primarily to the presence of the Colchester (No. 2 Coal) member horizon. These studies have shown no evidence of faulting in the area. Major reported faults in the mid-continent are tabulated in Table 2.5-2 and are shown on Figure 2.5-9.

The Colchester (No. 2 Coal) member, though a very narrow horizon, is extremely consistent and constitutes an excellent marker bed in the area. Subsurface mapping, based on a large number of site borings and numerous exposures in the area, shows very good stratigraphic continuity throughout the site area.

While it is theoretically possible that "minor" faults could remain undetected in the plant area, it can be stated, based on the extremely close stratigraphic control provided by the Colchester horizon, that no faults exist within the plant site area. In addition, no evidence of surface faulting was observed during excavation mapping. All geologic evidence indicates that there has not been any fault movement in the area during Pleistocene or recent time. Further, available information points to the inactivity of faults in the region of the site which extends well into the Paleozoic Era. Therefore, even assuming that faulting might exist in the vicinity of the site, it would present no hazard to Seismic Category I structures at the site owing to the fact that all geologic evidence suggests that any faults which may exist in the site area or environs are inactive.

Several faults have been inferred in the Galena Group in the Chicago metropolitan area (Reference 23). The original evidence upon which these faults were inferred to exist consisted of a number of seismic reflection lines run in conjunction with geotechnical studies for the Chicago Metropolitan Sanitary District.

Seismic studies provide indirect evidence of subsurface structure, and borings were done in several areas to attempt to confirm the existence of the these faults. These borings provided mixed results. In some cases, faults inferred from seismic data were not encountered or showed substantially smaller displacement than had been indicated. However, in a few cases, faults of small displacement were encountered in borings which had not been indicated in the seismic survey (Reference 52).

The faults in the Galena Group which are known to exist in the Chicago area can be dated as being post-Silurian and pre-Pleistocene in age. Though not present in the immediate area of the faults, the presence of Mississippian and Pennsylvanian blocks preserved in the nearby Des Plaines Disturbance (Reference 23) clearly indicates that similar deposits were laid down in the Chicago metropolitan area and subsequently eroded. Faulting caused by normal tectonic mechanisms very probably occurred during this extended period of deposition and erosion. The present bedrock surface is planed off and, owing to a lack of scarp development, does not suggest post-Pleistocene movement on the faults. Further, there is no evidence within the Pleistocene deposits to suggest recent movement.

Surface subsidence associated with underground mining occurs in localities above underground coal-mining activity. This may be reflected by only minor distortions or vertical displacements in the sediments above the No. 2 Coal. The possibility of future movements along these zones is extremely remote.

2.5.1.2.6 Solution Activity

The cores of bedrock recovered from 86 test borings were inspected in detail for evidence of rock solution activity which may be related to the development of voids. The boring logs illustrated in Figures 2.5-123 through 2.5-253 include the descriptions and distributions of voids observed in the rock cores. In this discussion, all openings in the rock described as vugs or porous zones have been combined into one category called rock voids. No solution channels were noted.

The rock voids observed in the cores were generally less than 1 inch in their longest dimension. Only a few 2-inch and 3-inch rock voids were observed. In all cases, the zones of core indicated as being porous were limited to isolated areas of less than 2 lineal feet of core, and the rock voids constituted 5% or less of the total rock volume within those zones.

Small rock voids were observed to some extent in many of the calcareous units penetrated by onsite borings. These units include the Silurian dolomite and the Ordovician-aged Fort Atkinson Limestone, the Scales Shale, and the Wise Lake-Dunleith Formations.

The Silurian dolomite was cored in Borings L-3 and L-4. Some porous zones with pinpoint to 0.1-inch openings were observed having a percent of voids estimated to be on the order of 1% to 2% of the total rock volume.

The basal Fort Atkinson contains scattered vugs with crystal deposits and numerous porous zones. In many instances, the porous zones contain small quantities of oil. Open spaces are confined to relatively narrow zones and never exceed 5% of the total rock volume, generally much less. Single openings in excess of 2 inches in diameter are rare.

The uppermost horizon of the Scales Shale consists of calcareous siltstone which is highly fossiliferous. Porous zones and cavities were observed in this unit at some locations. Most openings are lined with drusy crystalline surfaces and rarely exceed 0.5 inch in diameter. Open spaces are confined to relatively narrow zones having less than 5% voids.

In the Wise Lake-Dunleith Formations, local voids were observed at some localities, while none whatsoever had occurred at others. The highest percent of voids observed was approximately 5%. Individual openings rarely exceed 1 inch in diameter. This is stratigraphically the lowest unit penetrated by onsite borings. The top of this unit was encountered in the borings at elevations ranging from 267.1 to 343.4 feet.

In general, deep bedrock units contain a higher percentage of voids than the shallow units. The observed maximum percentage of voids at any location is 5%. This percentage is not considered significant from the standpoint of bedrock stability. No foundation problems are anticipated with regard to bedrock solution.

2.5.1.2.7 Man's Activities

There are no known instances of, or potential possibilities for, surface or subsurface subsidence, uplift, or collapse resulting from the activities of man within the site area. Former activities within the site vicinity have included underground and strip mining of coal. A detailed discussion of the coal mining is presented in Subsections 2.5.1.2.7.1 through 2.5.1.2.7.5. There are no large uses of groundwater nor any industrial disposal wells in this area. No surface subsidence due to groundwater withdrawals has been reported near the site.

The Natural Gas Pipeline Company of America operates two underground natural gas storage areas approximately 9 and 13 miles from the site. Both of the gas storage fields are associated with the Herscher Dome (see Figure 2.5-10). There have been no instances of uplift, subsidence, or collapse associated with these gas storage fields; therefore, no hazard is posed to the plant site due to the operation of these gas storage projects.

2.5.1.2.7.1 History of Coal Mining

Coal was first discovered in Illinois on 1679 by Father Hennepin, a missionary, who reported a "cole" mine on the Illinois River near the present-day town of Ottawa, approximately 34 miles west of the site. In 1810, coal was first mined in Jackson County in southern Illinois and shipped to New Orleans; sustained production was not achieved until 1833, when 6000 tons of coal were mined from the same locality and shipped to St. Louis.

Coal was accidentally discovered near Braidwood in 1854 on the farm of Thomas Byron (Reference 53). A coal bed 3.5 feet thick was encountered at a depth of 65 feet in a well which was being drilled for water. A company was formed, the well was enlarged to a shaft, and mining was begun the same winter. By the early 1880's, mining activity in the Braidwood area involved seven companies employing 2,180 men and producing 700,000 tons of coal annually. Within the area of interest surrounding the plant site (Figures 2.5-36 and 2.5-37), coal-mining development was closely related to activity in and near Braidwood. Mining of coal in this area can be divided into two distinct periods.

The first period, one of underground mining, began in the 1870's with the activities of the Eureka Mining Company and ended with the closing of the Wilmington Coal Mining and Manufacturing Company's Mine No. 6 at Torino in 1920. Production from underground mines declined from this time on, with the exception of the Number 3 Coal Corporation located in Section 23, T.31N., R.8E., which operated from 1927 through 1954. Upon abandonment of this long-lived producer, underground production in the area came to an end.

A second period of mining was begun in the 1920's with the development of large-scale earth-moving equipment which allowed strip-mining methods to supplant underground methods economically. Strip mining was begun near Braidwood in 1927, although it was not begun in the vicinity of the site until 1940, when the Wilmington Coal Mining Company began producing from a large pit centered in Section 28, T.32., R.8E. In 1947, the Northern Illinois Coal Company began operation in its nearby Pit No. 11 in Section 8, T.31N., R.9E., and produced until its interests were acquired by the Peabody Coal Company in 1956. From that time to 1974, coal was produced continuously from the Northern Mine (Figure 2.5-36, Numbers 24-30). The rate of mining was approximately 1 million tons per year, and this operation was the last producer in the area.

Total production of coal from the area shown in Figure 2.5-36 is estimated at over 26 million tons. Approximately 6.2 million tons was produced from underground mines, and about 20.5 million from strip mines. Details of the mines and their estimated production are shown in Table 2.5-5.

2.5.1.2.7.2 Coal Seams

The coal in the area of interest has been produced principally from the Illinois No. 2 Coal seam. The No. 2 seam is normally overlain by 30 or more feet of the Francis Creek Shale Member of the Pennsylvanian Carbondale Formation (Figure 2.5-19). This seam is also known as the Colchester Coal and in old reports as the "Third Vein." In the vicinity of the site, the No. 2 seam has a persistent thickness averaging very close to 3 feet.

A secondary producing seam, the No. 4, has been mined by Peabody Coal Company in its Pit No. 14 (Number 28 on Figure 2.5-36). In this area, the No. 4 seam lies approximately 57 to 64 feet above the top of the No. 2 seam and has an approximate thickness of 3 feet 8 inches. The No. 4 seam has been correlated with the Lowell Coal.

In the southwestern part of the area, thin seams of coal lie closely above and below the No. 2 seam. The upper seam is known as the "Cardiff Coal" (References 54 through 56) and has been mined together with the No. 2 seam in some of the underground operations. The geologic relationships of the minor coals in this area are not clear. The old records refer to various seams labeled as the No. 2A and the No. 2B, as well as an unidentified No. 3.

The eastern limit of the No. 2 coal in the vicinity of the site has been delineated by mining and drilling and is shown on Figure 2.5-36.

The coals in this area are classified as High Volatile C (Reference 57). Typical analyses are shown in Table 2.5-6.

2.5.1.2.7.3 Coal Mining Methods

Coal has been mined in the area of interest by two principal methods: underground by the "longwall" method, and on the surface by the variety of open-cut mining known as "stripping."

The area of interest surrounding the plant site lies within the First Mining District for coal mining as was promulgated by the State of Illinois. This area, shown on Figure 2.5-37, includes portions of Bureau, Grundy, Kankakee, Kendall, La Salle, Marshall, Will, Putnam, and Woodward Counties. Underground mining in this district utilized the "longwall advance system." Since this was the only domestic coal field during its period to produce any significant tonnage using the system, it was referred to and still remains known as the "Longwall District."

Underground longwall mining practice within the district consisted of sinking two shafts to the coal bed, one for hoisting and the other for ventilation. In Will County, depths of these shafts ranged from 70 to 125 feet. The shafts were protected by a "shaft pillar" ranging in size from a circular pillar with a 225-foot radius to a square pillar with a 60-foot side. Examination of the few underground maps available for the area of interest shows that local practice called for a shaft pillar some 400 feet long by 200 feet wide. Mining was advanced radially outward from the central pillar. All the coal was extracted, and no other pillars were left. Coal removed from each working face was transported along roadways which were supported by "pack walls." The pack walls were constructed of shale, siltstone, and clay mining wastes and were built 10 to 12 feet apart to allow for squeezing by roof pressures to permit an ultimate open width of about 9 feet. The roadways led to individual working faces having typical lengths of 42 feet (see Figure 2.5-38).

Each working face accommodated one or two miners. The coal was usually underlain by a bed of clay, which, when undercut from 8 to 12 inches, allowed the coal to fall under the influence of its own weight. Wedges were used to force any coal which did not fall freely. The broken coal was removed by shoveling or "mucking," placed in mine cars, trammed to the shaft, and finally hoisted to the surface. As the seams were generally about 3 feet thick, overhead rock was scaled off or "brushed" so as to provide a minimum head room of 4 feet for men and haulage mules. Excess rock and clay were either placed in the openings created by the removal of coal (referred to as "gob") or transported to the surface and placed in dumps known locally as "Red Dog Piles," the name derived from the red color which was generated from the oxidation of pyrite. Generally, 1 ton of dump material was hauled to the surface for each 3 tons of coal mined.

As the underground working faces were advanced, the partially supported ground left behind was allowed to subside gradually (Reference 58). Controlled subsidence transmitted the weight of the overlying ground onto the working face, which forced the coal to break when undercut. It was therefore important that the face be advanced uniformly so as not to create dangerously imbalanced ground pressures. Thus, in the longwall system, the roof of the mine was designed to subside to the floor in a controlled manner.

Past strip-mining practice in the vicinity of the site has entailed deep excavation with power shovels, draglines, and bucket-wheel excavators on a large scale. From 40 to 100 feet of overburden were removed to extract 3 feet of coal. As seen in the Peabody Coal Company's Northern Mine, Pit No. 11, (Figure 2.5-36), the overburden was removed in benches ranging in width from 50 to 200 feet and for lengths ranging from 2000 to 9000 feet. The overburden from each advance was placed upon the ground where coal had already been removed. A new area of coal was continually exposed and mined with the advance of the fresh face, or "highwall."

2.5.1.2.7.4 Coal Mine Locations and Production Data

Detailed investigation was made of available records from the Illinois Department of Mines and Minerals, the Illinois State Geological Survey, the Illinois Department of Highways, and the Peabody Coal Company. Land records in Will, Grundy, and Kankakee Counties were examined for ownership by mining companies. A field reconnaissance was made of the site area, and agricultural soil survey maps were examined for any evidence of mining activity. The results of the studies showing all known mines within approximately 1 mile surrounding the plant site and cooling pond areas are shown on figures 2.5-36 and 2.5-36A.

In the plant site area, borings were spaced on 100-foot centers in the area of Seismic Category I structures (Figures 2.5-16 and 2.5-33). The Colchester (No. 2 Coal) member was encountered in all the borings drilled, indicating that underground mining activity does not exist at the plant site. Also, coal development drill holes on approximately 330-foot centers (Reference 59) indicate that no underground coal mining underlies any portion of the NE1/4 and the SE1/4 of Section 19, T.32N., R.9E. Since the longwall mining system which was used in the district did not involve the use of isolated tunnels and drifts and allowed for complete extraction of the coal, the results of the development drill holes can be considered as a reliable indication that no mining has been pursued in the previously described parcels of Section 19. In the NW1/4 of this same Section 19, although land records indicate that in 1867 the Kankakee Coal Company held an interest in the E1/2 of the NW1/4, examination of the surface reveals that no mine shafts or dumps exist in this parcel, and that consequently no evidence exists that coal was ever mined in this quarter-section. The detailed topographic map presented in Figure 2.5-39 supports this conclusion.

The closest underground mine that exerts any influence upon the plant site is the Chicago, Wilmington and Vermillion Coal Company's "M" shaft in the SW1/4 of Section 19, T.32N., R.9E. (Figures 2.5-36 and 2.5-36a). This mine produced 277,845 tons of coal from the No. 2 seam between 1889 and 1891. Approximately 3 feet of coal was mined at a depth of 95 feet, or at about elevation 500. The calculated area of the underground mine workings computed from these production data is approximately 72 acres. This area is considerably less than the mine outline indicated in the mined-out coal area maps from the Illinois State Geological Survey, shown on Figures 2.5-36 and 2.5-36a. This undocumented outline is disproved by the development drilling; the alternative outline is the preferred interpretation.

In the E1/2 of the NW1/4 of Section 20, T.32N., R.9E., the Joliet Wilmington Coal Company mined a total of 150,363 tons from its No. 2 mine during the period 1905 to 1909. Drilling data have revealed the presence of a mined-out area of 31 acres. The published production data agree with tonnage computed from this area within 1%. These workings do not endanger the site.

In Section 17, T.32N., R.9E., records suggest that the Braidwood Coal Company may have mined coal underground at some time about 1879. An examination of Will County land records indicates that this company's involvement was limited to the NE1/4 of the SW1/4 of that section; it is therefore doubtful that any underground workings extend beyond this parcel. A strip mine was operated in the E1/2 of the NW1/4 of this same section about 1940, but the old workings as shown in the aerial photographs are not extensive and are estimated to have produced about 138,000 tons. These mines are not considered to pose any problems to the plant site.

In Section 18, T.32N., R.9E., records indicate that the Eureka Coal Company mined coal underground from two shafts and may have produced 180,000 tons of coal during the period 1872-1884. An examination of the Will County land records indicated that the interests of the Eureka Coal Company were limited to the NW1/4 of the section. As the mines in the area usually were mined on the basis of quarter-section land parcels, it is not likely that the underground workings of the Eureka mines extended south of the east-west center line of the section. Two mine dumps (Reference 60) occur in the N1/2 of this section and are believed to be the Eureka mines. As these two mines are more than 0.5 mile beyond the plant site boundary, they are considered to pose no problem to the integrity of the site. The only other evidence for mining in this section is a reference in the land records of Will County that the Kankakee Coal Company had an interest in the SE1/4 and in the E1/2 of the SW1/4 in 1869; however, a close examination of the surface of these lands did not reveal any mine dumps or any other evidence of mining. Consequently, it is considered that no mining was ever pursued in that parcel.

2.5.1.2.7.5 Surface Subsidence Due to Coal Mining

The extraction of coal from the old underground mines in the area outside of the plant site and cooling pond areas resulted in subsidence of the overlying land surface. Since the subsidence occurred directly over the old workings with a very limited lateral effect, the integrity of the plant site is not jeopardized.

Subsidence characteristics of longwall mines in this district have been studied in detail by the U.S. Bureau of Mines in cooperation with the Illinois State Geological Survey (Reference 61). Although the test mines studied lie some distance from the site, characteristics are so similar to those near the site that the same observations may be applied.

Surface indications of subsidence resulting from longwall mining are subtle and are frequently not visually perceptible (References 58 and 62). Longwall mining permits the complete extraction of the coal seam, which results in uniform subsidence over the entire mine area. The rates and magnitude of subsidence were partially controlled during mining by packwall construction and gob filling methods (References 58 and 62).

Maximum subsidence to be expected over mined-out areas near the site is largely a function of seam thickness removed and the depth of mining. For the No. 2 seam in the area of interest, a 3-foot thickness at a depth of burial of from 90 to 125 feet may be expected to produce a surface subsidence of less than 2 feet (Reference 62). Subsidence of the small magnitude is difficult to differentiate from natural variations in topographic relief. Some indications of subsidence can be observed in areas where rain-saturated soil on flat-lying farmland may delineate shallow sag ponds. Such indications are not sufficiently conclusive to identify the extent of underground workings.

The local characteristics of subsidence due to underground mining have been observed as follows (References 58, 61 and 63):

a. Uniform settling occurred over the extent of the mine workings.

b. The amount of vertical subsidence averaged 55% of the thickness of the coal seam to depths of about 200 feet (Reference 62).

c. The "angle of draw" (discussed in the following material) has been measured in the district at 8° from the vertical, sloping outward and away from the mine workings (Reference 61). This agrees with the typical angles of draw between 8° and 12° which have been observed worldwide in coal mines less than 300 feet deep (Reference 63).

D. Subsidence began immediately upon removal of the coal and ended within 2 or 4 years (Reference 61).

e. Where the coal seam was overlain by shale (as is typical in the area), abandoned mine openings became filled through progressive failure of the overlying shale.

f. Where the coal seam was underlain by "underclay" (as is also typical in the area), artifical supports, such as timbers and packwalls, were forced downward into the underclays by the weight of the burden. The underclay also would flow plastically upwards, tending to fill available openings.

Thus, for the typical underground mine in the area having a depth of 100 feet and a coal seam 3 feet thick, subsidence at the surface would be outside the vertical projection of the outer limits of the mine workings for a distance of less than 13 feet. Active subsidence would have come into equilibrium with the static ground load within about 4 years after mining ceased. Most openings can be considered either as having been "squeezed" together through plastic compression of packwalls and gob, or as having been filled through flowage of shale and clay. However, it can be expected that some unusually well supported mine workings may yet remain open.

The "angle of draw" is here defined to mean the angle between a vertical line from the edge of the mine workings and a line to the point where subsidence becomes negligible. The "limit of subsidence" on the ground surface is defined as the point where subsidence is less than 0.01 foot (Reference 63).

Based upon a review of worldwide coal-mine subsidence studies, the most severe angle of draw on record is 40.5°, which resulted from mining a series of 12 superimposed coal seams at depths to

5800 feet (Reference 63). For mines less than 200 feet deep, an angle of draw of 45° is considered safe and conservative. Assuming a maximum depth of mining adjacent to the plant of 118 feet, the closest safe distance of approach for any structure on the surface to the outer limit of the mine workings is 118 feet.

2.5.2 Vibratory Ground Motion

This subsection presents a discussion and evaluation of the seismic and tectonic characteristics of the Braidwood Station and the surrounding region.

The purpose of this section is to present the rationale used to develop the seismic design criteria for the Braidwood Station.

2.5.2.1 Seismicity

2.5.2.1.1 Seismicity Within 200 Miles of the Site

The North Central United States is among the areas of least seismic activity in the United States. Since this area has been populated for almost 200 years, it is likely that all earthquake events of Intensity VI or greater on the Modified Mercalli (MM) Scale (Table 2.5-7) which have occurred during this time span have been reported. Table 2.5-8 is a list of all known reported events which have occurred between 38° to 46° north latitude and 84° to 94° west longitude. The locations of these events and their spatial relationship to the area within a 200-mile radius of the site are shown on Figure 2.5-40. Within 200 miles of the site, 106 earthquakes have been known to occur. The largest were three events of Modified Mercalli Intensity (MMI) VII which occurred in 1909.

The locations of the events listed in Table 2.5-8 shown on Figure 2.5-40 which were located instrumentally are probably accurate to about ± 0.1°. The location of older events, not determined instrumentally, may have occurred as much as ± 0.5° from the stated location, as the reported epicentral locations for these events normally correspond to the locations of the nearest reporting population center.

There is no record of any event larger than MMI VII occurring within 200 miles of the site. If such an event had occurred, it is almost certain that it would either have been recorded in private journals or diaries or preserved in Indian legends as has been the case for other regions. The lack of such documentation indicates the absence of significant earthquake activity for a long period of time.

The most important earthquakes occurring within 100 miles of the site are as follows:

a. 1804, Fort Dearborn, Illinois, MMI VI - VII;

b. 1909, S. Beloit, Illinois, MMI VII;

c. 1912, northeastern Illinois, MMI VI;

d. 1972, northern Illinois, MMI VI.

Isoseismal maps of the above earthquakes have been constructed for all but the 1804 Fort Dearborn event. Those for the 1909 and 1912 events, which occurred approximately 55 and 25 miles from the site respectively, were prepared by J.A. Udden (References 64 and 65) and A.D. Udden (Reference 66) based on the Rossi-Forel scale of intensities which was in use at the time. These maps are reproduced here on Figure 2.5-41. The conversion to the Modified Mercalli Scale can be made using Table 2.5-7.

Little is known about the Fort Dearborn earthquake of 1804 beyond a report of "quite a strong shock" (Reference 67) because most of Chicago's early records were destroyed in the Great Fire.

The 1972 northern Illinois earthquake had an Intensity of VI, with its epicenter 35 miles south of the site (Figure 2.5-42). The shock was widely felt but did little damage (Reference 68).

Two other significant events occurred within 200 miles of the site: the July 18, 1909, central Illinois event and the September 27, 1909, southern Illinois event. The July 18 event was felt over an area of 35,000 mi^2 and was probably felt at the Braidwood site (Reference 69). The September 27 event occurred within the Wabash Valley and was probably felt at the site (References 69 and 70).

2.5.2.1.2 Distant Events

2.5.2.1.2.1 Central Stable Region

Within the Central Stable Region only one other event was recorded which may have been felt at the Braidwood site. This is the 1968 Intensity VII southern Illinois event (Figure 2.5-43) which occurred near Broughton, Illinois, approximately 225 miles from the site (Reference 69 and 70). This event occurred within the Wabash Valley area, an area noted for a relatively high frequency of events, the largest of which has been Intensity VII (Reference 71).

2.5.2.1.2.2 Mississippi Embayment Area

The largest recorded earthquakes which have occurred in the central part of the United States were the New Madrid events of 1811-1812. These events occurred in the Mississippi Embayment area of the Gulf Coast Tectonic Province (References 4, 71, and 72) at a distance of over 330 miles from the site (Table 2.5-9 and Figure 2.5-44).

Over a period of 3 months during 1811-1812, three large separate shocks occurred, the largest of which had an Intensity of XI-XII, as well as at least 250 minor events (References 73 and 74). There has been no recurrence of such a major earthquake in this zone, but there is evidence of activity prior to the New Madrid events. There is a report of a very large shock on December 25, 1699, with its epicenter in western Tennessee, which shook approximately the same area as the 1811-1812 events. Written records also indicate that "notably vigorous" shocks occurred in 1776, 1791 or 1792, 1795, and 1804. Indian traditions also record a previous earthquake which devastated the same area (Reference 73).

In addition to these events, an Intensity VIII event occurred in 1895 in Charleston, Missouri, also within the Mississippi Embayment area, which was probably felt at the Braidwood site.

2.5.2.1.2.3 Other Events

Two other events may have been felt at the Braidwood Station site: the 1886 Intensity X Charleston, South Carolina, event, which occurred in the Atlantic Coastal Province, and the 1935 Intensity VI Timiskaming, Canada event, which occurred on the Canadian Shield. Details of these and other distant events are presented in Table 2.5-9.

2.5.2.2 Geologic Structures and Tectonic Activity

The Braidwood site and the entire 200-mile radius site region lie within the Central Stable Region of the North American Continent (Reference 4). This region is characterized by a relatively thin veneer of sedimentary rocks overlying a crystalline basement. These areas were deformed principally by movements which occurred as a result of tectonic activity culminating in the late Paleozoic into a series of gentle basins, domes, and other structures. Since the end of the Paleozoic, the area has remained generally quiescent.

The site is located on the flank of the Illinois Basin near the Kankakee Arch. The most significant nearby structures are the Sandwich Fault Zone and the La Salle Anticlinal Belt. A description of these and other tectonic features in the area is presented in Subsection 2.5.1.1.4.

2.5.2.3 Correlation of Earthquake Activity with Geologic Structures or Tectonic Provinces

The Central Stable Region Tectonic Province is generally noted for its lack of significant seismic activity. To evaluate the earthquake potential of the Braidwood site, two different approaches were utilized to correlate earthquake activity with geologic structures and/or tectonic provinces. By the first approach, the 200-mile radius site region was subdivided into seismotectonic regions utilizing methods similar to those of

Reference 75. In the second approach, the site and its relationship to the Central Stable Region Tectonic Province and the Gulf Coastal Plain Tectonic Province was assessed, along with the relationship of the seismogenic structures of these provinces with the site.

2.5.2.3.1 Seismogenic Regions

Within 200 miles of the Braidwood Station, eight seismogenic regions can be delineated, primarily on the basis of structure. These subdivisions are also indicative of the differing geologic histories of the seismogenic regions and of their varying seismic histories.

The following is a description of the eight seismogenic regions within the 200-mile radius site area and other regions pertinent to the site. Each region is outlined on Figure 2.5-40.

2.5.2.3.1.1 Illinois Basin Seismogenic Region

The site is located on the north flank of the Illinois Basin Seismogenic Region. The northern and northeastern boundaries of this region correspond to and are defined by the limits of the Plum River and Sandwich Fault Zones.

The Braidwood Station lies just south of the Sandwich Fault Zone and just east of the Kankakee Arch, just within the Illinois Basin Seismogenic Region.

This region has experienced 60 recorded earthquakes, the largest of which were Intensity VI and Intensity VII. Some tentative correlation of events has been proposed by various authors, notably McGinnis and Ervin (Reference 76), who have postulated a correlation of earthquake events with areas of steep gradients in the earth's gravitational field which they interpret to indicate boundaries of crustal blocks. However, based on the present state of knowledge, these events are considered random. Therefore, the possibility of an Intensity VII event anywhere in the basin must be considered.

2.5.2.3.1.2 Ste. Genevieve Region

The Ste. Genevieve Region lies approximately 230 miles southwest of the site and is related to and defined by the imbricated Ste. Genevieve Fault Zone. This region exhibits a characteristic maximum intensity earthquake of MMI VI. While there is no evidence that this region and the included Ste. Genevieve Fault Zone are capable, fault plane solutions coincide with the trace of the fault (References 77 and 78). The boundary with the Illinois Basin is based on both a change in structure and by a contrast in seismicity.

2.5.2.3.1.3 Chester-Dupo Region

The Chester-Dupo Region, proposed by Nuttli (Reference 79), is defined by an area of faulting and folding in the vicinity of St. Louis. This region, approximately 175 miles southwest of the site, is one of moderate seismicity, with maximum events characteristic of MMI VI-VII. The boundary between this region and the Illinois Basin is marked by the transition from the folds and faults of this region to the deeper, structurally less complex Illinois Basin. This region marks a hinge line between the Illinois Basin and the front elements of the Ozark Uplift.

2.5.2.3.1.4 Wabash Valley Seismogenic Region

This seismogenic region is defined by the limits of the Fairfield Basin, the deepest part of the Illinois Basin, and by the northwest-trending faults of the Wabash Valley. The closest approach of this region to the site is approximately 155 miles. This area has moderate seismicity, with maximum events of MMI VII. Events occur more frequently in this region than events in the adjoining parts of the Illinois Basin (Reference 71). The boundaries of the Wabash Valley Seismogenic Region can be well defined by structure and geologic history as well as by its seismic pattern.

2.5.2.3.1.5 Iowa-Minnesota Stable Region

This region is one of extremely low seismicity, with a general maximum intensity of MMI V. The boundary between this region and the Illinois Basin is approximately 130 miles from the site and is marked by a gentle zone of flexure, the Mississippi River Arch.

2.5.2.3.1.6 Missouri Random Region

The Missouri Random Region is bounded by the Chester-Dupo Region to the east, and its contact with the Illinois Basin Region is marked by the Lincoln Fold. This region lies approximately 175 miles southwest of the site. This area is characterized by the occurrence of random seismic events of maximum MMI V which are not associated with any known structure.

2.5.2.3.1.7 Michigan Basin Region

The Michigan Basin Region is an area of extremely low seismicity, with a total of 10 recorded events, the largest an MMI VI. This area is separated from the Illinois Basin by the Kankakee Arch and lies approximately 80 miles northeast of the site.

2.5.2.3.1.8 Eastern Interior Arch System Seismogenic Reqion

This region is composed of a series of gentle Paleozoic arches and domes within the eastern part of the Central Stable Region. Structurally, this area is composed of the Wisconsin, Kankakee, Findlay, and Cincinnati Arches and the Wisconsin and Jessamine

Domes. While this system can be subdivided into the various structures, the geological history of the structures and lithologies as well as general patterns of seismicity are similar. Since the boundaries between any of the structures are rather nebulous, divisions would be rather arbitrary.

The Wisconsin Dome in the northern part of the Central Stable Region consists of Precambrian rocks and is therefore more reflective of the Laurentian Shield subdivision of the Central Stable Region than the Interior Lowlands, the subdivision within the United States (References 4 and 72). The Wisconsin Dome is an extremely stable part of the Central Stable Region and represents the most seismically stable part of this region, with maximum seismic activity of MMI V.

The Wisconsin Arch is defined structurally by the low, northsouth-trending, uplifted area extending south from the Wisconsin Dome and is herein defined as including the east-west-trending crosscutting folds and faults of southern Wisconsin.

The boundary between the Wisconsin Arch and the Kankakee Arch is extremely hard to define. The name changes from the Wisconsin Arch to the Kankakee Arch northeast of Kankakee, Illinois. The Wisconsin Dome and Arch have a Precambrian core and are believed to have acquired their relief primarily by uplift, whereas the relief on the Kankakee Arch is due primarily to more rapid subsidence of the bordering basins.

The arch system continues southeastward to join the Cincinnati Arch and the Jessamine Dome. The Findlay arch is a northeastward splay off the Cincinnati Arch and separates the Michigan Basin from the Appalachian Basin.

Seismicity within this region is generally of MMI V. However, isolated events of MMI VII have occurred which cannot be related to specific structures. Therefore, the entire region must be assigned a maximum potential random event of MMI VII.

2.5.2.3.1.9 Anna Seismogenic Region

The Anna Region is at the intersection of the Kankakee, Findlay, and Cincinnati Arches in western Ohio. This area has experienced continued and moderately severe seismic activity. The largest historic earthquakes commonly have been of Intensity VII, with a single event of a maximum Intensity VII-VIII. This region is defined as lying within a basement structural zone bounded on the south by a northwest-trending band of basement faulting, on the east by a zone of structural weakness marked by a north-south-trending band of magnetic highs and lows, on the north by a change from igneous extrusive to igneous intrusive rock, and on the west by the change from acidic extrusive to basic extrusive rocks (Reference 80). The combination of geological features within this area is unique. There is no other area within the central United States with the combination of factors similar to this region. The earthquake events which have occurred in this region are not random but rather the result of the unique combination of geological phenomena (Reference 80).

2.5.2.3.1.10 New Madrid Seismogenic Region

One of the most important seismogenic zones for determining maximum possible ground motion within the central United States is the New Madrid Seismogenic Region. This zone can be defined approximately on any tectonic map as corresponding to the northern portion of the Mississippi Embayment, which is the northern portion of the Gulf Coastal Plain Tectonic Province (Figures 2.5-40 and 2.5-45; References 4, 71 and 72).

The New Madrid events of 1811-1812 were the largest earthquakes ever experienced in the central and eastern United States. Chimneys were knocked down as far north as St. Louis, Missouri, and the aftershocks from these events continued for 2 years (Reference 67). These events occurred more than 330 miles from the Braidwood site. Extensive studies have been conducted to determine the northernmost region in which these events could occur.

This has been documented in a Sargent & Lundy and Dames & Moore report dated May 23, 1975 (Reference 81). Further discussion on this matter took place at a meeting held on January 26, 1976, in the offices of the Illinois State Geological Survey, Urbana, Illinois at the request of Public Service of Indiana. Representatives were present from the Nuclear Regulatory Commission, the Illinois State Geological Survey, the Indiana Geological Survey, the Kentucky Geological Survey, St. Louis University, Sargent & Lundy, Dames & Moore, and Seismograph Service Corporation (Birdwell Division). The scientific data presented clearly indicated that the New Madrid area, at the intersection of the Pascola Arch and the Ozark Dome, is tectonically unique and that the northernmost extent of the structurally complex New Madrid area is conservatively taken as 37.3° N and 89.2° W, or 275 miles from the site. It remains the applicant's interpretation, based on tectonic, geophysical and seismic data, that New Madrid-type events should not extend across tectonic province boundaries and up the Wabash Valley Fault System.

More recent geophysical and seismological data also support the applicant's position. Interpretations of gravity and magnetic data in Illinois (References 82 and 83) and others support the view that the Rough Creek Fault Zone separates distinct crustal provinces.

A regional microearthquake network has recently been installed in this area. Analysis of data obtained from this network indicates that the New Madrid region and the Wabash Valley Region are two distinct seismic regimes (Reference 84).

2.5.2.3.2 Tectonic Provinces

2.5.2.3.2.1 Central Stable Region Tectonic Province

The Central Stable Region is noted for its general lack of significant seismic activity, with the largest events generally of MMI VII.

Within this tectonic province there are several zones of relatively high activity. These are (1) near Attica, New York, (2) near Anna, Ohio, (3) the Wabash River Valley of southern Illinois and Indiana, (4) in eastern Kansas and Nebraska along the midcontinent gravity and magnetic high in the area of the Nemaha Anticline, and (5) near St. Louis, Missouri (Figure 2.5-45).

The Attica events are associated with the Clarindon-Lindon Structure, and the August 12, 1929, event has been assigned an Intensity of VIII by Coffman and von Hake. However, the amount of damage and estimated magnitude of this event indicate that it was probably Intensity VII-VIII, and that the assigned intensity of VIII is extremely conservative (Reference 85).

The area around Anna, Ohio, has experienced a relatively large amount of seismic activity compared to other areas of the Central Stable Region. As described previously, the area of earthquake activity corresponds to a highly complex Precambrian structural zone. In addition, the March 8, 1937, event, which has been assigned an Intensity VII-VIII by Coffman and von Hake (Reference 69), has been analyzed, and all indications are that this event had a maximum epicentral intensity of VII (Reference 80).

The Wabash Valley Fault Zone was described in Subsection 2.5.2.3.1.4 and has had maximum recorded seismic activity of Intensity VII.

The area along the midcontinent gravity and magnetic high (in the area of the Nemaha Anticline) has had several events of Intensity VII, and the relationship of earthquake activity to the midcontinent gravity and magnetic high has been documented in Subsection 2.5.2 of the Wolf Creek PSAR (Reference 86).

The activity near St. Louis, Missouri, has been assigned to the Chester-Dupo Region as defined by Nuttli (Reference 79) and documented in the PSAR for the Callaway Plant (Reference 87). Historical activity in this area has had a maximum Intensity VI-VII.

In addition to these areas of the Central Stable Region which have had relatively high seismic activity, an Intensity VIII event was reported in the Keweenaw Peninsula of Michigan in 1906 (Reference 69). The area of the epicenter is highly faulted, and the areas of damage and preceptibility correspond to areas of mining activity. Smaller events which occurred earlier in the year as well as the larger event of 1906 all appear directly attributable to mining activity (Reference 88). The felt area of the 1906 event was approximately equal to that for an average Intensity III-IV event (Reference 69).

2.5.2.3.2.2 Gulf Coastal Plain Tectonic Province

The New Madrid events of 1811-1812 did not occur in the Central Stable Region Tectonic Province, but in the Gulf Coastal Plain Tectonic Province. These events are associated with a highly complex structural zone near the crest of the Pascola Arch (see Subsection 2.5.2.3.1.10).

If these events are translated to the closest approach of this tectonic province to the site, they could be expected to occur no closer than 275 miles from the site or 55 miles closer to the site than the 1811-1812 events occurred.

2.5.2.3.3 Earthquake Events Significant to the Site

By both methods of analyzing the tectonic association of earthquake events with structure, as described previously, the most significant earthquakes in the region are the 1909 Intensity VII Beloit earthquake, the 1972 Intensity VI northern Illinois earthquake, the 1912 Intensity VI northeastern Illinois earthquake, the 1804 Fort Dearborn earthquake, and the New Madrid earthquakes of 1811-1812. This evaluation is based on epicentral intensity, felt area, distance from the site, and tectonic association.

2.5.2.4 Maximum Earthquake Potential

Based on the discussion in Subsection 2.5.2.3, the maximum earthquake which could be expected would be an Intensity VII event equivalent to the occurrence of an event similar to the 1909 Beloit Intensity VII event near the site. This is equivalent also to the occurrence of the largest event which has ever been recorded within the Central Stable Region, and which cannot yet be associated with a specific structure or structural region; it is therefore described as random. The level of ground motion experienced from a near field Intensity VII event would envelope the motion expected from a recurrence of a New Madrid-type event at the closest approach of the Mississippi Embayment, a distance of 275 miles from the site.

2.5.2.5 Seismic Wave Transmission Characteristics of the Site

The engineering properties of the soils and bedrock units at the site were evaluated using field geophysical measurements and laboratory testing; the properties determined by laboratory testing are discussed in Subsection 2.5.4.2.2.

Geophysical investigations performed at the plant site are presented in Subsection 2.5.4.4. The velocity of compressional and surface wave propagation and other dynamic properties of the natural subsurface conditions were evaluated from these investigations, and the data were used in analyzing the response of the materials to earthquake loading.

Dynamic moduli for the subsurface soil and rock at the site were calculated based on measured properties. The in situ field measurements were compared with laboratory tests on the same materials. These analyses are presented in Subsection 2.5.4.7.

Seismic wave velocities and densities for the deeper rock strata in the region have been measured by others (Reference 85). These data confirmed field measurements and were used in studies of site dynamic behavior.

2.5.2.6 Safe Shutdown Earthquake

The recommended safe shutdown earthquake (SSE) was defined as the occurrence of an Intensity VII event near the site. This near field event may generate a maximum horizontal ground acceleration of 0.13g (Reference 89). However, at the time of the review of the construction permit application, the NRC considered the occurrence of an earthquake of Intensity MM VIII to be equally probable (a low order of probability) at any place in the eastern Central Stable Region. The NRC also took the position that, based on the postulated occurrence of an intensity MM VIII at the site, a safe shutdown earthquake of 0.20 at the bedrock-till interface was adequately conservative for the Braidwood Station. For purposes of licensing, this value was applied at the foundation level. Utilizing the subsurface properties presented in Subsection 2.5.4.7, the corresponding ground surface acceleration was found to be 0.26g. This would be the controlling seismic event even if a New Madrid-type event were postulated to occur at Vincennes, Indiana, more than 155 miles from the site. This conclusion is based on information presented in Reference 80.

The ground response spectra prepared following the guidelines of Regulatory Guide 1.60 for a horizontal ground acceleration of 0.26g are presented on Figure 2.5-47.

2.5.2.7 Operating-Basis Earthquake

The operating-basis earthquake (OBE) is intended to indicate those levels of ground motion which could reasonably be expected to occur at the plant site during the plant operating life.

On the basis of the seismic history of the area, it appears very unlikely that the site will be subjected to any ground motion of significant levels during the life of the nuclear power station. It is probable that the maximum level of ground motion experienced at the site during historic time was Intensity VI and was due to the 1909, Intensity VII, Beloit earthquake. For this intensity, the maximum horizontal ground acceleration at the site can be postulated to be on the order of 0.06g. Therefore, the OBE acceleration at the bedrock surface was conservatively recommended to be 0.06g for horizontal ground motion.

A probability analysis (Reference 90) of the occurrence of earthquakes at the station was also performed using the data on past earthquakes in the area.

In performing this analysis, epicenters were assumed to occur at random in a 195,000-mi^2 area around the station. The results of this probability analysis show that a site Intensity of MMI VI has an average return period of 2150 years. Because of this long return period, the site intensity of VI was selected conservatively as the OBE. For purposes of licensing of the plant, however, the acceleration level for the OBE was selected at 0.09g. It should be pointed out that this acceleration level is higher than the level of acceleration expected for an Intensity VI event and corresponds approximately to acceleration levels expected for an Intensity VI-VII event (Figure 2.5-4; Reference 89). Additional conservatism was then used as the 0.09 g acceleration level was applied at foundation levels utilizing the subsurface properties presented in Subsection 2.5.4.7. The resulting maximum horizontal ground acceleration at the ground surface was 0.13g.

The response spectra for 0.13g horizontal ground acceleration prepared following the guidelines of Regulatory Guide 1.60 are presented as Figure 2.5-48.

2.5.3 Surface Faulting

No evidence for surface faulting was noted at the site or the area surrounding the site. The nearest known major surface fault in the region is the Sandwich Fault Zone; its nearest approach is approximately 10 miles north of the site.

Based on the data contained in Subsections 2.5.1 and 2.5.2, and the interpretation and conclusions from those data, there are no capable faults within 5 miles of the site, as defined in Appendix A to 10 CFR 100, January 1977.

There are no known capable faults in the regional area (200-mile radius around the plant site).

2.5.3.1 Geologic Conditions of the Site

A discussion of the lithologic, stratigraphic, and structural conditions of the site and the area surrounding the site, including its geologic history, is contained in Subsection 2.5.1.

2.5.3.2 Evidence of Fault Offset

There is no evidence of fault offset at or near the ground surface at the site. The structural geology at the site and surrounding region is discussed in Subsections 2.5.1.1.4 and 2.5.1.2.5.

2.5.3.3 Earthquakes Associated with Capable Faults

There have been no historically reported earthquakes within 5 miles of the site. No capable faulting is known to exist within 200 miles of the site.

2.5.3.4 Investigation of Capable Faults

No capable faulting is known to exist within 200 miles of the site.

2.5.3.5 Correlation of Epicenters with Capable Faults

No capable faulting is known to exist within 200 miles of the site, and no earthquake epicenter is known within 5 miles.

2.5.3.6 Description of Capable Faults

No capable faulting is known to exist within 200 miles of the site.

2.5.3.7 Zone Requiring Detailed Faulting Investigation

Since geologic investigations of the site have not indicated evidence of capable faulting, the detailed fault investigation required for a capable fault is not needed.

2.5.3.8 Results of Faulting Investigation

Geologic investigations of the site and the area surrounding the site have indicated that no capable faulting is present within 200 miles of the site and that no surface faulting is present within 5 miles of the site; a study of surface faulting is therefore not required.

2.5.4 Stability of Subsurface Materials and Foundations

This subsection presents an evaluation and summary of the geotechnical suitability and stability of the subsurface materials to support the plant foundations. A general site plot plan is shown on Figure 2.5-16.

2.5.4.1 Geologic Features

A detailed discussion of the geologic characteristics of the site is given in Subsection 2.5.1.2. A comprehensive field and laboratory investigation program including borings, water pressure testing, piezometers, test pits, geophysical surveys, field reconnaissance, detailed mapping of the excavation, and various static and dynamic laboratory tests was undertaken to determine the geologic features at the site and their significance with relation to site suitability and stability.

A discussion of jointing is presented in Subsection 2.5.1.2.5.1. Discussions of faulting and solution activity are presented in Subsections 2.5.1.2.5.3 and 2.5.1.2.6, respectively. Discussions of man's activities and surface subsidence due to coal mining are presented in Subsections 2.5.1.2.7 and 2.5.1.2.7.5, respectively.

2.5.4.2 Properties of Subsurface Materials

This subsection presents an evaluation of the static and dynamic properties of the various soil and rock strata encountered at the site. These values are based upon:

a. a review of all field and laboratory tests performed during this investigation,

b. a review of the geophysical surveys performed during this investigation,

c. a review of the latest available literature, and

d. a review of similar studies made recently for nuclear generating plants at other locations.

2.5.4.2.1 Field Tests

Field test results are presented in Subsections 2.5.4.3 and 2.5.4.4.

2.5.4.2.2 Laboratory Tests

Tests were conducted on soil samples obtained using the Dames & Moore Type U soil sampler, the Osterberg piston sampler, and a 4-inch-diameter double-tube core barrel. Test results on the undisturbed samples are in good agreement with the results obtained using the Dames & Moore Type U sampler.

Representative soil samples and rock cores extracted from the test borings were subjected to laboratory tests to evaluate the physical characteristics of the soil and rock encountered at the site. The laboratory program, performed under the direction of Dames & Moore, included the following tests:

a. static tests:

    1. direct shear,

    2. unconfined compression (soil and rock),

    3. triaxial compression,
    4. consolidation,
    5. moisture and density determinations,
    6. grain size analysis,
    7. Atterberg limits,
    8. compaction characteristics, and
    9. permeability.

  b. dynamic tests:
    1. cyclic triaxial compression, and
    2. resonant column.

The testing program is considered adequate to define the range of strength and engineering characteristics to be expected in each stratum. The soil profile within the plant area is sufficiently well defined to justify interpolation between points where laboratory data were obtained. Since there will be very little natural soil under Seismic Category I structures, extensive testing of natural soils in these areas was not warranted.

2.5.4.2.2.1 Static Tests

2.5.4.2.2.1.1 Direct Shear Test

The results of the direct shear tests and the corresponding moisture contents and dry densities are presented in Table 2.5-10 and on the boring logs. The method for performing direct shear testing is described on Figure 2.5-56 (Sheet 2).

2.5.4.2.2.1.2 Unconfined Compression Tests

2.5.4.2.2.1.2.1 Unconfined Compression Tests on Soil

The results of the unconfined compression tests on soil and the corresponding moisture contents and dry densities are presented in Table 2.5-11 and on the boring logs (Figures 2.5-123 through 2.5-247). The method of testing is described on Figure 2.5-56 (Sheet 1).

2.5.4.2.2.1.2.2 Unconfined and Unconsolidated Undrained Compression Tests on Rock

The strengths of the underlying rock formations were evaluated by unconfined compression tests on representative rock core samples. The tests were performed by the Robert W. Hunt Company and Walter H. Flood and Company, Inc., both of Chicago, Illinois, in accordance with the standard testing procedures of ASTM D2938-1971. Samples approximately 4 inches in height and 2 inches in diameter were subjected to a constant rate of axial load. The results of the rock compression tests are presented in Table 2.5-3.

2.5.4.2.2.1.3 Triaxial Compression Tests

The results of the unconsolidated undrained (UU) and consolidated undrained (CU) triaxial compression tests on soil and the corresponding moisture contents and dry densities are presented in Tables 2.5-11 and 2.5-12 respectively and in Figure 2.5-57. The method of testing is described on Figure 2.5-56 (Sheet 1).

2.5.4.2.2.1.4 Consolidation Tests

Consolidation tests were performed on representative soil samples to determine the compressibility characteristics of the soils. The method of performing consolidation tests is described on Figure 2.5-58. The results of the consolidation tests are presented in Figure 2.5-59. The consolidation test results from 4-inch-diameter undisturbed cored samples and samples obtained with the Dames & Moore Type U sampler generally agree with the expected variation due to the variability of the soils.

The consolidation curves for cohesive soils generally indicate some degree of sample disturbance. It is believed that the disturbance can be attributed to the fact that most of the samples were obtained with a drive (Dames & Moore Type U) sampler. Since the cohesive soils are overconsolidated, the soils are very brittle and therefore quite sensitive to disturbance during sampling and sample preparation.

Detailed examination of the data indicates that the effects of sample disturbance are most evident during initial loading below the preconsolidation pressure and become relatively minor in the virgin range. The true in situ behavior below the preconsolidation pressure is therefore better represented by the rebound branch of the curve or an unload-reload cycle initiated at or near the preconsolidation pressure.

2.5.4.2.2.1.5 Moisture and Density Determinations

In addition to the moisture and density determinations made in conjunction with the strength tests and consolidation tests, independent moisture (ASTM D2216) and density tests were performed on other soil samples for correlation purposes. The results of all moisture and density determinations are presented to the left on the boring logs (Figures 2.5-123 through 2.5-247). The moisture and density results from samples obtained with the Osterberg sampler (cohesionless soils) and core sampler (cohesive soils) generally agree well with the results from samples obtained with the Dames & Moore Type U sampler.

2.5.4.2.2.1.6 Grain Size Analysis

Grain size distributions were determined for representative soil samples to aid in classification and correlation of the physical soil properties. The particle size analyses were conducted in accordance with the standard procedures of ASTM D422-1963. The results of these tests are presented on Figures 2.5-94 and 2.5-118.

2.5.4.2.2.1.7 Atterberg Limits

Atterberg limit tests were performed on selected samples of cohesive (fine-grained) soils encountered in the test borings. The tests were performed in accordance with the standard testing procedures of ASTM D423-1966 and ASTM D424-1959. The Atterberg limits, consisting of the liquid limit, the plastic limit, and the resulting plasticity index, were determined to facilitate classification of the soils according to the Unified Soil Classification System and for correlation purposes. The results of the Atterberg limit tests and the plasticity indices are presented to the left on the boring logs (Figures 2.5-123 through 2.5-247).

2.5.4.2.2.1.8 Compaction Characteristics

Modified Proctor moisture density relationships were determined for representative samples of the onsite granular soils obtained from the borings in order to evaluate their suitability for use as compacted fill. The tests were conducted in accordance with the standard test method of ASTM D1557-1970. The method of testing is described on Figure 2.5-60. The results of the Modified Proctor compaction tests are presented on Table 2.5-13 and Figure 2.5-61.

Relative density tests were performed on representative samples of coarse-grained (sandy) soils obtained from test pits and borings in the plant site area. In situ moisture and density of the sands were obtained in the field and laboratory, and minimum and maximum densities were determined in the laboratory by the standard test method of ASTM D2049-1969. The results of these tests are presented in Table 2.5-13.

Test results indicate that the maximum density obtained by the method of ASTM D2049-1969 are 0.0 to 5.0 pounds per cubic foot higher than maximum densities obtained by the method of ASTM D1557-1970. The variation in the test results is normal and in agreement with published data comparing the two ASTM density determination methods.

2.5.4.2.2.1.9 Permeability

Permeability test results and a discussion of these results are presented in Subsection 2.5.6.2.5.1.

2.5.4.2.2.2 Dynamic Tests

Results of cyclic triaxial compression tests and resonant column tests are presented and discussed in Subsection 2.5.4.7.

2.5.4.3 Exploration

The surface and subsurface field exploration programs consisted of the following:

a. geologic reconnaissance and excavation mapping,

b. test borings,

c. piezometers,

d. test pits, and

e. geophysical surveys.

2.5.4.3.1 Geologic Reconnaissance and Excavation Mapping

A program of geologic field reconnaissance was conducted by Dames & Moore at the Braidwood site and is discussed in Subsection 2.5.1.2.1.

2.5.4.3.1.1 Excavation Mapping Program

2.5.4.3.1.1.1 Introduction

Geologic mapping of the excavations for the power block structures at the Braidwood Station was performed by geologists from Sargent & Lundy to confirm the stratigraphic and structural relationships of the units underlying the site, and to verify that the site stratigraphy as exposed in the excavations was in agreement with that determined by the boring program and presented in the PSAR. The mapping program started on February 25, 1976, and ended on March 3, 1976. The descriptions of the stratigraphic units and the contacts between the units were verified by the Illinois State Geological Survey during a site visit on March 11, 1976 (see FSAR Attachment 2.5A). The main excavation was also inspected by the NRC on April 20, 1976, and the findings are presented in FSAR Attachment 2.5B.

Geologic sections of the exposed strata were prepared and correlated to the boring logs in order to incorporate additional detail in the descriptions of the various lithologies and stratigraphic relationships for the FSAR. The sections were prepared using controlled field mapping and photography.

2.5.4.3.1.1.2 Field Procedures

In the slopes cut into soil, shallow trenches were opened at approximately 400-foot intervals using hand tools in order to expose the undisturbed soil. The spacing between trenches reflects the general uniformity of the soil strata across the excavation. Geologic sections were made using a 5-foot Jacob's Staff and a Brunton Compass. Separate stratigraphic units and contacts between units were described in the sections. After setting up control points to provide coordinates and elevations within each section, the trenches were photographed.

In the excavation walls cut into rock, geologic sections were prepared at 50-foot or larger intervals, depending on variation in lithology vertically and horizontally. Separate stratigraphic units and contacts between the units were described in these sections, which were measured using a 100-foot engineer's tape. After setting up control points to provide coordinates and elevations within each section, the walls were photographed.

A total of 38 control points was set at various stratigraphic horizons throughout the excavation. At 29 of the control point locations, correspondingly numbered geologic sections were measured (5 in soil, 24 in rock). The remaining nine control points were used to mark the top of rock. Between the time that the control points were set and the time that they were surveyed, six control points were destroyed by construction. Of these, three were top of rock points and three were geologic section points (two soil sections and one rock section).

Photographic coverage was used in the excavation mapping. Approximately 190 photographs were taken at 145 locations. A 5 foot scale and a photo location number were provided in most of the photographs for reference. Many of the photographs were overlapped to provide continuous photo mosaics of selected walls and slopes.

The soil-bedrock interface was marked with control points on 100-foot or larger intervals on the slopes along the perimeter of the excavations for Seismic Category I structures.

2.5.4.3.1.1.3 Stratigraphy Within the Excavation

The sequence of stratigraphic units exposed within the main plant excavation at the Braidwood site is Pleistocene-age Parkland Sand, Equality Formation, and Wedron Formation underlain by the Pennsylvanian-age Carbondale Formation.

The stratigraphy encountered within the Braidwood Station excavation is the same as that encountered in the site borings. During the excavation mapping, those joints which were observed were generally tight, with no apparent movement along the joint surface.

The stratigraphy within the Braidwood Station excavation is represented by photographs (Figures 2.5-49, 2.5-51, and 2.5-53), geologic sections (Figures 2.5-50, 2.5-52, 2.5-54, 2.5-297 and

2.5-299), and location maps (Figures 2.5-55, 2.5-296, and 2.5-298).

2.5.4.3.2 Test Borings

Thirty widely spaced geologic borings were drilled at the site from August 1972 to January 1973 by Soil Testing Services, Inc., under the supervision of Dames & Moore. Sixty-nine additional borings were drilled at the plant area from January 1973 through March 1973 by Raymond International, Inc., under the supervision of Dames & Moore. Twenty-two borings were drilled at the ultimate heat sink area and are discussed in Subsection 2.5.6. Detailed descriptions of the soil and rock encountered in the borings are presented on Figures 2.5-123 through 2.5-253. The soils were classified according to the Unified Soil Classification System described on Figure 2.5-27. A summary of the borings is given on Table 2.5-14. The site borings range in depth from 35.5 to 345.0 feet below the ground surface and were drilled at the locations shown on Figure 2.5-16. The purpose of the borings was to obtain samples for the determination of the details of lithology, structure, and physical properties of the subsurface strata at the site.

The drilling was done with truck-mounted rotary wash equipment. Drilling mud and/or casing was used in the soil portion of the borings. Bedrock coring was performed with the aid of water. All borings drilled were grouted with cement. Rock was cored utilizing both NX and HQ (wireline) double-tube core barrels, which provide rock cores of approximately 2 inches and 2-1/2 inches in diameter respectively.

Soil samples suitable for laboratory testing were obtained using a Dames & Moore Type U Sampler. The sampler is 3-1/4 inches in outside diameter and approximately 2-1/2 inches in inside diameter as shown on Figure 2.5-62. Soil samples were also extracted utilizing a standard split-spoon sampler approximately 2 inches in outside diameter and 1-3/8 inches in inside diameter. These samples were taken using the Standard Penetration Test procedure. Additional sampling of the lacustrine sands was done with the Osterberg piston sampler to obtain undisturbed samples for determining in situ density and dynamic properties. The Osterberg samples were 3 inches in diameter.

Undisturbed samples of the glacial till soils, approximately 4 inches in diameter, were obtained by coring with a double-tube core barrel.

Selected borings were water-pressure-tested as they were being drilled in rock. A single inflatable packer was used to isolate the bottom 10-foot section of the drill hole each time that the core barrel was removed. The tests normally consisted of two pressure levels and one repeat of the lowest pressure level. Maximum net water pressure used was 1.0 psi per foot of depth.

The results of the pressure tests are presented on the boring logs (Figures 2.5-123 to 2.5-253) as ranges for each interval tested and as lugeons which were computed according to the following formula:

$$\text{Lugeons} = \frac{1820 \text{ x rate of loss (gpm)}}{\text{Interval tested (ft) x net pressure (psi)}} \qquad (2.5\text{-}1)$$

The lugeon is defined as 1.0 liter of water loss per meter of hole per minute. Equation 2.5-1 determines Lugeons, so that units in this equation should not be expected to cancel to liters/meter/minute.

The net pressure is given as:

$$\text{net pressure} = \text{gauge pressure + column pressure} - \text{friction loss} \qquad (2.5\text{-}2)$$

The column pressure is equal to the depth to the upper packer or the depth in feet to groundwater, whichever is smaller, times a constant of 0.433 psi/ft of depth (hydrostatic pressure gradient).

2.5.4.3.3 Piezometers

Thirty-nine piezometers have been installed at the Braidwood site. Summaries of the depths and water levels are presented in Tables 2.5-15 and 2.5-16, respectively. The locations of these piezometers are shown in Figure 2.5-16. The piezometers installed at the essential service cooling pond are discussed in Subsection 2.5.6.

2.5.4.3.4 Test Pits

Thirteen test pits were excavated within the site area for the purpose of performing in-place density tests and obtaining bulk samples for laboratory relative density tests. Locations were chosen primarily to obtain representative samples of coarsegrained soils. The locations of these test pits are shown on Figure 2.5-16, and the logs of the test pits are shown on Figures 2.5-254 through 2.5-260. Test pits excavated at the essential service cooling pond are discussed in Subsection 2.5.6.

2.5.4.3.5 Geophysical Surveys

Geophysical surveys conducted at the site are discussed in Subsection 2.5.4.4.

2.5.4.3.6 Geologic Cross Sections

Geologic cross sections showing foundation elevations for Seismic Category I structures are presented in Figures 2.5-25, 2.5-26, 2.5-92, and 2.5-93.

2.5.4.4 Geophysical Surveys

The following site geophysical surveys were conducted:

a. a seismic refraction survey to estimate the depth of soil overburden and to evaluate the compressional wave velocities of the bedrock and overburden,

b. a surface and shear wave survey to determine surface wave types and characteristics and to study shear wave velocities of near-surface materials,

c. an uphole velocity survey to define compressional wave velocities further,

d. a downhole shear wave survey to evaluate shear wave velocities of the overburden soils and bedrock,

e. ambient noise studies to determine the predominant frequencies of ground motion of the site due to background noise levels, and

f. geophysical borehole logging to assist with stratigraphic correlation.

The geophysical surveys were performed in the plant site area at the locations shown on Figure 2.5-64. The seismic parameters derived from the geophysical surveys are generally applicable, as the geologic profiles are essentially identical beneath all structures. The lateral variations in compressional wave velocities shown for each layer are not significant when determining the engineering properties of the soils and bedrock below Seismic Category I structures.

A description of each phase of the geophysical explorations is provided in the following paragraphs along with a summary on Figure 2.5-63 which represents the velocity-depth model for the site.

2.5.4.4.1 Seismic Refraction Survey

A seismic refraction survey was conducted to evaluate the subsurface characteristics of the site, and to confirm the nature of the underlying strata as established by cross sections based on geologic borings. The survey was conducted at the site along two seismic lines for a total length of 4000 lineal feet. The seismic lines were oriented approximately north-south and eastwest and intersected at Boring A-6 near the approximate center of the site as shown on Figure 2.5-64.

Seismic energy was produced by the detonation of a small explosive charge on both machine-drilled and hand-dug holes. The energy released by the detonations was picked up by vertically oriented geophones fitted with a spike for coupling with the underlying soil. Hall-Sears geophones (4.5 hertz) were spaced at 50-foot intervals along the seismic refraction lines.

The seismic energy was recorded by a 24-channel Dresser S.I.E. RA-44 seismic amplifier coupled with a Dresser S.I.E. R-24A recording oscillograph and a 12-channel Electro-Tech Labs ER-72-12A seismograph.

The geophysical field crew consisted of two geophysicists, an operator, a licensed powderman, a helper, and a driller and helper. The field work was performed from September 25 to October 2, 1972.

Compressional wave velocities and the depths to various subsurface layers under the site were evaluated by plotting the first arrival times of the seismic energy at each geophone station against the distance of each geophone from the shot point. The time-distance data from each profile are shown on Figures 2.5-65 and 2.5-66. To evaluate the effect of topography on the interpreted layers, one section (seismic line 1A, station 0+00 to station 10+00) of time-distance data was corrected to a 600-foot elevation datum. The segment of seismic line 1A corrected for topography is presented in Figure 2.5-67. In addition, profiles of the various subsurface layers are shown directly below each corresponding time-distance plot. The depths for these profiles are computed from the time-distance plots by using the time intercept method of calculation. A summary of these calculations is presented in Table 2.5-1. This table shows the depths below the surface at each shot point for each interpreted velocity layer and its corresponding compressional wave velocity. In using the time-distance plots, note that the information was compiled from shot points at several locations along the seismic line. For clarification of the figures, two plot symbols have been used to indicate the origin of the geophysical shots: from the left (.) and from the right (+). In addition, the apparent compressional wave velocities (slope of each line) are shown above each line segment. The subsurface section shown represents an evaluation of the most probable conditions based upon interpretation of presently available data. Some variation from these conditions must be expected.

The geophysical refraction survey indicates that four zones of contrasting seismic velocity can be detected. The compressional wave velocities for these zones are summarized on Figures 2.5-65 through 2.5-67. There were occasional indications on the seismic refraction lines of high velocities within this layer. The Birdwell three-dimensional logs indicate an average velocity of about 12,000 fps from 122 to 142 feet in depth. This layer is underlain by a layer with a velocity of 16,000 to 17,000 fps. The 12,000-fps layer does not appear on the seismic refraction lines as a first arrival. This layer is too thin and of insufficient velocity contrast to the layers above and below it. The critical distance for the refraction arrival of this layer is longer than the critical distance for the refraction arrival from the 16,000 to 17,000-fps layer below it. This is the classic case of a hidden layer without velocity inversions. The lowest refractor, represented by the velocity range of 16,000 to 17,000 fps, apparently represents a velocity change within the Fort Atkinson Limestone.

The velocities shown on the seismic profiles below each time-distance plot represent the best estimate of the true velocities for the corresponding section of profile. These velocities were obtained by an averaging technique applied to the apparent velocities as shown on the time-distance plots. The lateral changes in the velocities are considered reasonable based on the lithologic changes observed in the boring logs. The most consistent layer to be interpreted is the Fort Atkinson Limestone.

In comparing the segment of seismic line 1A that was computed to a flat datum to the other profiles, it can be seen that the two interpretations are in close agreement for both layering and velocities. The slight variation (25 feet) in the top of the Fort Atkinson Limestone was not considered significant to justify applying this method of computation (i.e., datum corrections) to the remainder of the profiles. The difference in the top of the Fort Atkinson Limestone could be due to the presence of the high-velocity zone, and not the interpretive techniques. This high-velocity zone within the Carbondale Formation has been shown as a dashed line on the seismic profiles. The only indication of the presence of this zone was from the information on the Birdwell logs.

2.5.4.4.2 Surface Wave and Shear Wave Velocity Survey

In order to evaluate further the dynamic bedrock characteristics, a surface wave and shear wave velocity survey was conducted by Geoterrex, Ltd. in the vicinity of the plant site. The survey was conducted along a 2400-foot section trending northeast/southwest as shown on Figure 2.5-64.

Surface and shear wave velocities were computed from measurements recorded by two 3-component Sprengnether Engineering Seismograph seismometers in conjunction with a Dresser S.I.E. R-24A recording oscillograph. The Sprengnether seismometers were placed 350 feet apart in the vicinity of Boring A-3, and explosives were detonated at varying distances ranging from 1000 feet to 2400 feet from the nearest seismometer.

The surface waves generated at this site by small explosions at a shallow depth are relatively small in amplitude. Three surface waves were observed in this study. The characteristics of these waves are given in Table 2.5-18.

The surface waves observed during this study all have predominant motion in the longitudinal and transverse directions, with very little or no motion in the vertical direction. The site has a characteristic frequency range of 9.5 to 13.5 hertz. Significant amplification of seismic energy will probably occur only within this frequency range.

2.5.4.4.3 Uphole Velocity Survey

An integrated uphole velocity survey in Boring A-2 was performed by Dames & Moore to provide a check on the compressional and shear wave velocities measured during the seismic refraction surveys. The boring was cased to 50 feet below the ground surface with 4-inch diameter casing.

The compressional wave velocity survey was completed by burying small explosive charges at depths of 3 to 3-1/2 feet and at a distance of 25 feet from the boring. The seismic response to the explosive charges was detected in the boring with a 12-trace geophone cable (velocity cable) and recorded on an Electro-Tech Labs ER-72-12A seismograph.

The compressional wave velocity data obtained from the Birdwell Log was integrated by summing the reciprocal velocities for each 1-foot interval. This integration was adjusted for total travel time by data obtained from the Dames & Moore uphole survey. The results of this and the uphole compressional wave velocity survey are presented on Figure 2.5-68. The dashed line on the figure represents a best-fit curve obtained from the uphole survey test data, while the solid line represents a best fit obtained from the Birdwell integration.

It can be seen that the compressional wave velocities measured from this survey differ from the compressional wave velocities measured in the seismic refraction surveys. The seismic refraction survey measures and averages the compressional wave velocities over a longer distance, whereas the uphole velocity survey measures the compressional wave velocities at an isolated point (Boring A-2).

A portion of the Fort Atkinson Limestone between the depths of 154 and 164 feet in Boring A-2 was not detected in the standard Dames & Moore uphole survey because of the geophone spacing, but this formation is quite apparent on the integrated survey. The additional velocity values for units below the Fort Atkinson Limestone are also shown on the integrated survey, although they were not resolved by the refraction survey.

2.5.4.4.4 Downhole Shear Wave Survey

The downhole shear wave survey was performed by Dames & Moore utilizing Boring A-2. A three-component low-frequency geophone (Mark Products LI-3D-S) was lowered into the boring.

Energy was introduced into the ground by striking the vertical face of a shallow excavation (reinforced with a wooden plank) at the top of the boring with a 10-pound hammer. The seismic response of the energy was detected in the boring by a three-component, low-frequency geophone (Mark Products LI-3D-S) and was recorded on the Dresser S.I.E. system. Multiple recordings were made of the seismic energy at 10-foot depth intervals. This data was reduced and plotted as a time-depth curve and is shown on Figure 2.5-69.

In addition to this method, explosive charges were fired at distances of 1,000 to 2,000 feet away from the boring. The resultant seismic energy was detected by the same geophone and recorded on the Dresser S.I.E. system. Recordings were made of the seismic energy at successive 25-foot intervals.

The results of both of these techniques are summarized on Figure 2.5-63. These data are referenced to the subsurface conditions at Boring A-2. This summary represents the seismic model for the site.

This seismic model was used to compute arrival times, which were compared directly to the records that were produced as a result of firing explosive charges into Boring A-2. The results of this technique provided only verification of the compressional wave velocities, in that the shear wave arrivals were difficult to classify.

2.5.4.4.5 Ambient Vibration Measurement

Measurements of the ambient background motion of the site and its response to natural motion generators are indicative of the dynamic properties of the site. These measurements were made by Dames & Moore at the three locations shown on Figure 2.5-64 during relatively quiet periods of no noise or ground activity.

A three-component, direct-writing, Sprengnether Engineering Seismograph, Model VS-122, was used for recording ambient ground motion. The seismograph has gain characteristics in the velocity mode of 20, the acceleration mode of 12, and the displacement mode of 200. A VS-1100D amplifier with a gain characteristic of 100 was coupled to the seismograph in all recordings. The resulting maximum gain level for velocity is 2,000, for acceleration, 1,200, and for displacement, 20,000. The three components of ground motion measured were radial, vertical, and transverse. The observed characteristic frequencies at the site in radial, vertical, and transverse directions ranged between 4.5 and 25.0 hertz.

Location 2, Boring A-3, appears to be the quietest location, while Location 1, Boring A-6, appears to be the least quiet location. Results of the ambient ground motion measurements are presented in Table 2.5-19.

2.5.4.4.6 Geophysical Borehole Logging

All borings were logged by Dames & Moore with the Widco Porta-logger upon completion of drilling. The purposes of the geophysical logging were to confirm the presence of the No. 2 coal seam throughout the site, to assist in the identification of lithology, and to assist with stratigraphic correlation.

The geophysical logs obtained included both gamma ray and single electrode resistance profiles. Coal seams are characterized by an extremely low reading on the gamma ray log, accompanied by a high reading on the electrical resistance log. A detailed description of the Widco Porta-logger including a discussion of its capabilities and limitations is included on Figure 2.5-70. The results of the geophysical logging with the Widco Porta-logger are shown on the geophysical logs of borings A-1 through A-11, P-3, P-6, P-10, and L-1 through L-4.

The Birdwell Division of Seismograph Service Corporation was contracted to run 3-dimensional velocity log, density log, and caliper log surveys in boreholes A-1 and A-2. The results as presented on Figure 2.5-71 were used to supplement information obtained from the surface geophysical survey summarized in Figure 2.5-63.

2.5.4.5 Excavations and Backfill

2.5.4.5.1 General

Excavations in soil and rock were required to achieve foundation grade for the plant structures. The excavations extended through the Parkland Sand, Equality Formation, Wedron Formation, and into the Carbondale Formation sandstone and siltstone. Surficial sands (Parkland Sand and Equality Formation) were cut on slopes of approximately 2:1 horizontal-to-vertical. Excavations within the till (Wedron Formation) were cut on slopes of approximately 1:1 horizontal-to-vertical. Excavation slopes within rock were nearly vertical. A quality control program was followed for all excavation and backfill operations at the site.

The criteria for blasting used for rock excavation at the Braidwood Station is covered in Sargent & Lundy Specification L-2714, entitled "Preliminary Site Work." A minimal amount of blasting was required for excavation of the plant foundations. Only eight blasts were used, all occurring between December 31, 1975 and January 22, 1976. No concrete was in place for any structures at the time of the blasts. The blasts were monitored at the site boundaries using seismographic tests to ensure that no damage was caused to residential structures. Blast data for the eight blasts are presented in Table 2.5-44. The majority of the plant foundations were excavated using conventional construction techniques such as ripping and ram-hoe methods.

Pittsburgh Testing Laboratory provided the inspection and performed in-place density tests on the compacted backfill to verify that the required compaction was obtained. The excavation, placement, compaction and testing operations were continuously monitored by a soil engineer.

2.5.4.5.2 Main Plant

2.5.4.5.2.1 Excavation

Excavation for the main plant was carried to final grades within the soil and upper rock by using heavy construction equipment. Blasting was required for excavations in the competent rock. The depth of the excavation varied throughout the main plant site. The excavation extended to a minimum depth required to remove all eolian and lacustrine sand deposits. This depth was approximately 20 feet below final grade. The maximum depth of excavation was 84 feet under portions of the auxiliary building. The locations and limits of excavations for the main plant including Seismic Category I structures are shown in plan (Figure 2.5-72) and section (Figure 2.5-73). The excavated sand was stockpiled east of the main plant. The excavated topsoil, till, and rock were disposed of at designated locations on site.

The final subgrade surfaces of all major structures were protected against frost, ponding of water, and construction activity until the protective mud mat was poured.

Excavation dewatering was accomplished by constructing a slurry trench around the excavation limits. The location of the slurry trench is shown in Figure 2.5-74.

2.5.4.5.2.2 Backfill

The backfill material used consisted of sand previously excavated from the main plant site, the circulating water pipeline corridors, and from approved borrow areas east of the main plant as indicated in the project specifications. In addition, lean concrete was used in lieu of sand backfill adjacent to the containment building walls beneath the fuel handling building. The locations and limits of backfill are shown in plan (Figure 2.5-75) and section (Figure 2.5-76).

The sand backfill within the zone of significant influence of loadings produced by the main plant structures was placed in horizontal lifts and compacted by use of vibrating rollers to a minimum of 85% relative density as determined by ASTM D2049-69. The backfill was placed according to Sargent & Lundy specifications and was monitored by a soil engineer. The sand backfill within the remaining areas was placed in horizontal lifts and compacted by use of vibrating rollers to a minimum of 80% relative density as determined by ASTM D2049-69.

The static and dynamic properties of the sand backfill are discussed in Subsections 2.5.4.2 and 2.5.4.7.

The envelope of the 58 grain size curves for the backfill material within the zone of significant influence of loadings produced by the main plant structures is shown on Figure 2.5-261. Laboratory relative density tests were performed on representative samples of the sand backfill material that was placed within the zone of significant influence of the loadings in the main plant area. The minimum test densities range from 80.1 to 91.0 pcf; the maximum test densities ranged from 103.5 to 114.5 pcf.

A total of 273 in-place density tests (ASTM D-1556) were performed on the sand backfill compacted to 85% relative density.

The frequency of field density and laboratory testing exceeded the minimum specified. Material testing requirements are shown in Table 2.5-45. With the exception of two in-place density tests, the in-place field densities for the backfill ranged from 104.3 to 125.7 pcf, with the relative densities ranged from 85.2% to over 100%. Of the two tests which failed to reach the minimum requirement of 85% relative density, one test is in the area beneath the earth ramp constructed for the reactor placement in Unit 1. When the reactor was in place and the ramp was removed, the area of the failing test was retested. The other test that failed to meet the minimum relative density was accepted on the basis of an in-place density greater than 95% of the modified Proctor compaction test, ASTM D1557. The in-place density was 99.7% of the modified Proctor compaction test.

Laboratory testing of lean concrete test specimen sets were made during the placement of the concrete material backfill in the main building structure area. During the backfill placement, an evaluation of backfill was performed. The results of the evaluation are shown in Table 2.5-46. It can be concluded that results of the compressive strength tests performed for the lean concrete used beneath and surrounding Category I building structures indicate that the actual strength is higher than the design strength.

The average actual ultimate bearing pressure of the lean concrete used for Category I building structures exceeds the ultimate bearing capacity of the founding strata.

2.5.4.5.3 Pond Screen House

2.5.4.5.3.1 Excavation

Excavation was carried to final grade using heavy construction equipment. The excavation for the Pond Screen House extended into the Wedron silty clay till to a depth of 36 feet below final grade. The location and limits of excavation for the Pond Screen House are shown in plan (Figure 2.5-16) and section (Figure 2.5-25). The excavated eolian and lacustrine sand was stockpiled for reuse as backfill. The excavated topsoil and till was disposed of at designated locations on the site.

The final subgrade surface was protected against ponding of water and construction activity until the protective mud mat was poured. Excavation dewatering was accomplished by constructing a slurry trench around the excavation limits and is discussed in Subsection 2.5.4.6. A plan showing the location of the slurry trench is given in Figure 2.5-74.

2.5.4.5.3.2 Backfill

The backfill material used consisted of sand excavated from approved borrow areas east of the main plant. It was placed and compacted under the same criteria and controls as the plant area backfill described in Subsection 2.5.4.5.2.2. Continuity of the perimeter dike slurry trench cutoff was accomplished by constructing the slurry trench up to and in contact with the Pond Screen House walls on both the east and west sides of the Screen House.

2.5.4.5.4 Seismic Category I Pipelines

2.5.4.5.4.1 Excavation

The essential service cooling water pipelines consist of the makeup pipeline and the discharge pipeline. The makeup pipeline extends from the pond screen house to the power block. The discharge pipeline extends from the power block to the essential service cooling water discharge structure near the south end of the essential service cooling pond, as shown in Figure 2.5-74.

The makeup and discharge pipelines occupy the same excavation from the power block to nearly the pond screen house. A geologic section for the pipelines in this shared excavation is presented in Figure 2.5-5.

Excavation for the essential service cooling water pipelines between the power block and pond screen house was carried to final grade within the till by using heavy construction equipment. Blasting was required for rock excavation. The excavated eolian and lacustrine sand was stockpiled west of the plant. The excavated topsoil, glacial till, and rock were disposed of at designated locations on site. The depth of the excavations for the Seismic Category I pipelines extended into the Wedron silty clay till and rock and varies along the route. The plan and geologic sections for the pipelines are shown in Figures 2.5-16 and 2.5-25.

Excavation dewatering between the power block and pond screen house was accomplished by construction of a slurry trench around the excavation limits and is discussed in Subsection 2.5.4.6. A plan showing the location of the slurry trench is given in Figure 2.5-74.

The essential service cooling water discharge pipeline, between the point where it separates from the makeup pipeline and the bend in the pipeline at approximately 52+00S, 43+55E, was constructed in the same manner as the portion from the power block to the pond screen house. The excavation for the remainder of the essential service cooling water discharge pipeline was opened during the fall and winter of 1978 exposing the glacial till. As a result of the excavation being exposed to weathering during the winter, a revised method of supporting the pipelines was initiated. The revised method consisted of concrete support pads spaced to allow a maximum twenty-foot clear span for the pipe. Removal of the disturbed till was carried out locally to ensure each support pad was resting on undisturbed till.

2.5.4.5.4.2 Backfill

Within the excavation for the main plant, sand backfill was used to support the pipeline. This sand backfill was placed and compacted using the procedures similar to those described in Subsection 2.5.4.5.2.2 for the main plant backfill. Above the pipeline, sand backfill was placed in horizontal lifts and compacted by using vibrating rollers or hand tampers to a minimum of 80% relative density as determined by ASTM D-2049-69.

The envelope of the three grain size curves for the sand backfill around the pipeline within the main plant excavation is shown in Figure 2.5-262. A total of 13 in-place density tests (ASTM D-1556) were performed on this material with a minimum relative density of 86.3%. The minimum requirement was 85% relative density as determined by ASTM D2049-69.

Results of the in-place density tests for compacted granular fill placed outside the main plant area for buried pipeline indicate compliance with project specifications. The envelope of the 12 grain size curves for essential service water pipeline backfill within the essential service water cooling pond is shown in Figure 2.5-309.

From the limits of the main plant excavation to the bend at plant coordinate 52+00S, 43+55E, bash was used for support of the Seismic Category I pipelines. Bash is a mixture containing cement, fly ash, sand, and water. After opening the pipeline excavation, the pipelines were supported on bash pads to facilitate welding. The entire excavation was then completely filled in with bash to an elevation one foot above the top of the piping, thereby completely encasing the pipeline in bash. Above this level, the sand backfill was placed and compacted as previously described for the pipeline within the main plant excavation.

South of the bend at 52+00S, 43+55E, the pipelines were encased in lean concrete. The excavation was backfilled with sand placed in lifts and compacted with vibratory rollers. In-place dry density tests for the backfill yielded relative density values greater than or equal to 85%.

Between the main plant and lake screen house the Seismic Category I essential service water supply pipe (ESWS) and non-safety-related circulating water supply pipe are buried in a common trench. As indicated above, the ESWS pipelines are founded on Wedon silty clay till and are backfilled with bash to the top of the pipes. Figure 2.5-25 shows a profile along the pipeline alignment. The top of the till is above the top of the pipes in most areas and in all cases is above the pipe centerline. The till and bash will not erode if the circulating water supply pipes should break as the result of an SSE event.

Laboratory testing of lean concrete and/or bash test specimen sets for buried piping were made during the placement of the concrete material backfill. During the backfill placement, an evaluation of backfill was performed. The results of the evaluation are shown in Table 2.5-46. The average actual ultimate bearing pressure of the lean concrete placed under the ESWP exceeds the ultimate bearing capacity of the founding glacial till. The average actual ultimate bearing pressure of the lean concrete used as a backfill material exceeds the ultimate bearing pressure of the compacted granular fill. The minimum 28-day compressive strength for the pipeline backfill was 180 psi, which exceeds the specified value of 150 psi.

Table 2.5-48 summarizes details of pipeline subgrade and backfill materials. Exact excavation limits and cross-sections are not provided.

2.5.4.5.5 Ultimate Heat Sink

Excavation for the ultimate heat sink is discussed in Subsection 2.5.6.

2.5.4.6 Groundwater Conditions

Groundwater conditions at Braidwood Station, including the permeability of the various hydrogeologic units underlying the plant site, a history of groundwater level fluctuations, and gradients in the site vicinity, are discussed in Subsection 2.4.13.2. Laboratory and field permeability test results are presented in Tables 2.5-20 through 2.5-24. Groundwater levels measured in piezometers installed during site investigations are presented in Table 2.5-25. Groundwater levels during construction, measured in eight observation wells around the main plant excavation (locations are shown on Figure 2.4-36) are presented graphically in Figure 2.4-44. Groundwater levels were also measured in seven observation wells around the cooling lake (locations are shown on Figure 2.4-37) and are shown in Figure 2.4-45.

For the design of safety-related plant structures, the groundwater level was assumed to be at plant grade, elevation 600 feet MSL. All subsurface and foundations are designed to withstand full hydrostatic loads.

The groundwater monitoring program is described in Subsection 2.4.13.4.

2.5.4.6.1 Excavation Dewatering

Excavation dewatering was accomplished using slurry trench cutoffs constructed around the excavation limits for the main plant, the pond screen house, and the Seismic Category I pipeline corridor, as shown in Figure 2.5-74. A cement-bentonite slurry trench cutoff was constructed around the main plant excavation, and a soil-bentonite slurry trench cutoff was constructed around the pond screen house and the Seismic Category I pipeline corridor. The slurry trench cutoffs were 2.5 feet wide and extended to an approximate depth of 2.0 feet into silty clay till. Seepage through the slurry trench cutoffs and the underlying glacial till was minor and, along with direct precipitation, was removed from the excavations with sump pumps.

2.5.4.7 Response of Soil and Rock to Dynamic Loading

2.5.4.7.1 General

This subsection presents analyses of the responses of the rock, in situ soil and recompacted soil to dynamic and seismic loading conditions; design values for dynamic response analyses; and seismic design criteria for major structures.

2.5.4.7.2 Dynamic Tests

Dynamic tests on soil and rock samples include dynamic triaxial compression tests and resonant column tests.

The following parameters were developed in the dynamic studies of soil and rock:

a. Young's modulus of elasticity (E);

b. modulus of rigidity (G)

E and G are related by

$$E = 2G (1 + \mu)$$

where μ is the Poisson's Ratio of the soil; and

c. a damping factor.

A generalized summary of the dynamic moduli and the damping for the subsurface materials at the site is presented in Table 2.5-26. The dynamic moduli of elasticity and rigidity were evaluated from the results of geophysical measurements and laboratory tests.

In order to illustrate the variation of the dynamic soil properties, modulus of rigidity (shear modulus) and hysteretic damping, with the single amplitude shear strain, summary plots of all laboratory soil test data are presented in Figures 2.5-77 through 2.5-80. The design curves shown on these figures generally represent the mean values of the test results as determined by the method of least squares and represent the relationships utilized during design. In the case of the till (Figures 2.5-79 and 2.5-80), the higher modulus and damping values were not utilized to arrive at the mean values, since it is considered that they are not representative of the entire soil mass in situ.

2.5.4.7.2.1 Dynamic Triaxial Compression Tests

The behavior of representative soils under dynamic loading was evaluated by conducting dynamic triaxial compression tests. The tests were performed by Professor M. L. Silver in the Soil Mechanics Laboratory at the University of Illinois, Chicago Circle Campus. The cohesive soil samples were tested at field moisture content and density. One 4-inch diameter undisturbed sample obtained by coring was tested for comparison with samples obtained with the Dames & Moore Type U sampler. The results from samples obtained by the different sampling procedures were very similar. Although some variations in results occur, the variation is no greater than would be expected considering sampling disturbance, laboratory preparation, and natural variation in material properties. The test results are summarized in Tables 2.5-27 and 2.5-28 and are presented graphically on Figure 2.5-81.

2.5.4.7.2.1.1 Sample Preparation

2.5.4.7.2.1.1.1 Granular Soils

To prepare samples for testing, a preweighed amount of dry sand was vibrated into a membrane-lined 2.4-inch-diameter mold in five equal layers to a height required to achieve the desired relative density. The cap was placed on the sample and affixed to the membrane with 0-rings, and vacuum was applied to keep the sample from deforming while the mold was removed and while micrometer measurements of the sample diameter and height were obtained. The triaxial cell was assembled around this sample, a small confining pressure was applied, and the vacuum was released, allowing water to saturate the sample by capillary action. The required value of confining pressure was then applied with backpressure to ensure saturation. Saturation of the samples was checked by measuring Skempton's B coefficient. In all tests, the B coefficient is approximately 1.0. The samples were allowed to consolidate isotropically under confining pressures representative of in situ conditions.

2.5.4.7.2.1.1.2 Cohesive Soils

Cohesive soil samples, obtained from both the Dames & Moore sampler (2.4-inch diameter) and by coring (approximately 4-inch diameter) were obtained to evaluate the dynamic characteristics of the cohesive soils. To prepare the Dames & Moore samples for dynamic material property tests, the soil was first extruded from the brass rings and placed in a mitre box, where the ends were trimmed square. The average diameter and initial height and weight of the sample were recorded and the sampled density was calculated. The triaxial cell was assembled around the sample. The cored samples were prepared in the same way except that it was not necessary to extrude the samples, since they had been shipped in a waterproof package. The confining pressure was then applied with backpressure to ensure saturation. In all tests, the B coefficient was in excess of 0.9.

2.5.4.7.2.1.2 Laboratory Procedure and Data Analysis

Material property tests were performed under controlled strain conditions. To begin the test, a very small amplitude sine wave signal (0.5 hertz) was programmed into the loading frame. The piston was connected to the load cell, the recording equipment was zeroed, and the sample was cycled at the lowest possible strain amplitude. At the tenth load cycle, the pen of the x-y recorder was lowered to record the load-deformation hysteresis loop for modulus and damping calculation. The tenth load cycle was chosen for modulus and damping determination as representative of the duration of strong motion for the safe shutdown earthquake postulated for the site (Reference 91). At the end of cycle 25, the test was stopped and the drainage valve was opened to allow dissipation of pore pressure. The drainage valve was again closed, a new, slightly higher strain amplitude was programmed, and another test was performed. The procedure was repeated six or seven times for each sample giving a record of dynamic sample response covering the range between approximately 0.01% to 1.0% single-amplitude axial strain.

Values of dynamic Young's modulus (E) were determined by measuring the slope of the line connecting the extreme points of the hysteresis loops obtained at the tenth load cycle. The same loop was used to calculate the hysteretic damping, using the equation

$$\lambda \frac{1}{2\pi} \frac{\Delta W}{W} \qquad (2.5\text{-}3)$$

where ΔW is the total dissipated energy per cycle as represented by the area of the hysteresis loop, and W is the work capacity per cycle (References 92 and 93).

The value of Poisson's ratio required for these calculations was estimated, since accurate measurement of Poisson's ratio is difficult to accomplish experimentally.

For cohesionless soils, the modulus has been found to be related to the confining pressure by the following equation.

$$G = 1000\ K_2\ (\bar{\sigma}_m)^{1/2} \qquad (2.5\text{-}4)$$

where K_2 is a soil parameter and $\bar{\sigma}_m$ is the mean effective principal stress which for triaxial tests is equal to the effective confining pressure. The influence of strain amplitude on the modulus can thus be expressed through its influence on the parameter K_2.

For cohesive soil, the wide variations in dynamic soils properties are often taken into account by normalizing the shear modulus (G) with respect to undrained shear strength (SS) and expressing the relationship G/S_u as a function of shear strain.

2.5.4.7.2.2 Resonant Column Tests

Dynamic torsional shear (resonant column) tests were performed on 15 representative soil and rock samples to evaluate the modulus of rigidity of these materials. The method of performing resonant column tests is described on Figure 2.5-82. The tests were conducted over a range of confining pressures. Tests on samples of cohesive soils and rock were conducted at field moisture content and density. The cohesionless soil was recompacted in the laboratory to a relative density of at least 80% and saturated prior to testing. The results of the resonant column tests are presented in Table 2.5-4.

2.5.4.7.3 Field Seismic Surveys

The geophysical explorations were performed at the plant site. A detailed description of the seismic material properties for each stratum under the site and the methods used to determine these properties are contained in Subsection 2.5.4.4.

2.5.4.7.4 Soil-Structure Interaction

A detailed description of the soil-structure interaction analyses for the plant structures is contained in Subsection 3.7.2.4.

The dynamic soil properties which are used in the analysis are shown in Figures 2.5-77 through 2.5-80.

2.5.4.8 Liquefaction Potential - Main Plant

A comprehensive liquefaction evaluation was performed for the sands in the ultimate heat sink area (see Subsection 2.5.6). These studies indicated that the in situ granular soils, which are similar to the in situ granular soils at the power block, were stable against liquefaction under the safe shutdown earthquake. However, for additional conservatism, the granular soils below the power block were excavated, and the onsite soils were placed and compacted to a minimum relative density of 85%. It is therefore concluded that the foundation material at the power block area has ample margin of safety against liquefaction.

2.5.4.9 Earthquake Design Basis

2.5.4.9.1 General

This subsection provides a summary of the derivation of the OBE and SSE and a summary of the earthquake selection for liquefaction and seismic response analysis of earthworks.

2.5.4.9.2 Safe Shutdown Earthquake

A detailed discussion of the SSE can be found in Subsection 2.5.2.6.

The recommended safe shutdown earthquake was defined as the occurrence of an Intensity VII event near the site. This near field event would produce horizontal bedrock accelerations of 0.12g (Reference 89).

In view of the NRC staff position, the effects of a random Intensity VII-VIII occurring near the site have been investigated. This event would result in a maximum horizontal ground acceleration of 0.20g (Reference 89). As an additional means of conservatism this value has been applied at foundation level. Utilizing the subsurface properties presented in Subsection 2.5.4.2, the corresponding ground surface acceleration was found to be 0.26g.

2.5.4.9.3 Operating-Basis Earthquake (OBE)

A detailed discussion of the derivation of the OBE is given in Subsection 2.5.2.7.

The operating-basis earthquake is intended to indicate those levels of ground motion to which plant structures might realistically be subjected during their economic life.

On the basis of the seismic history of the area, it appears unlikely that the site will be subjected to any ground motion of significant levels during the life of the nuclear power station. It is probable that the maximum level of ground motion experienced at the site during historic time was due to the 1909 Intensity VII Beloit earthquake and was Intensity VI at the site. For this condition, the maximum horizontal ground acceleration on rock at the site was probably on the order of 0.06g (Reference 91).

A probability analysis (Reference 90) of the occurrence of earthquakes at the station was performed using the data on past earthquakes in the area and the available information on the attenuation of intensity over the distance between the earthquake location and the site. The results of this probability analysis show that a site Intensity of VI on the Modified Mercalli scale has an average return period of 2150 years. Because of this long return period, the site Intensity of VI was selected conservatively as the operating-basis earthquake. However, to expedite licensing of the plant, the acceleration level for the OBE was selected as 0.09g. The 0.09g accleration level was conservatively applied at foundation level, resulting in a maximum horizontal ground acceleration of 0.13g.

2.5.4.10 Static and Dynamic Stability

2.5.4.10.1 Main Plant

2.5.4.10.1.1 Settlement

The plant structures at the Braidwood Station are founded on overconsolidated till, bedrock, or compacted granular fill. For the small pressure reduction caused by excavation, the rebound of the bearing strata is negligible.

The compressibility of the foundation subgrade materials was evaluated from the results of laboratory compression tests and static soil and rock properties summarized in Table 2.5-26. All static settlements were computed using the tangent modulus method after Janbu (Reference 94). The tangent modulus is expressed:

$$M = mP_a \, (P' / P_a)^{1-a} \qquad (2.5\text{-}5)$$

where

M = tangent modulus (tons/ft^2),

m = Modulus number,

a = Stress exponent,

P' = Effective vertical stress (tons/ft^2), and

P_a = reference pressure (1 ton/ft^2).

Due to sample disturbance, in situ behavior was assumed best represented by the first unload-reload cycle of the consolidation curve near the preconsolidation pressure. The results of field and laboratory tests are presented in Subsection 2.5.4.2. The resulting parameters are listed in Table 2.5-3. Of the rock units, only the Carbondale, Spoon, and Brainard Formations are compressible enough to be of engineering importance. The geological profile used in the analyses is presented in Subsection 2.5.4.3.6. The foundation loads and the maximum total settlements under static loading for plant structures are summarized in Table 2.5-29. Most of the total settlement will occur as the structures are constructed. The estimated differential settlements on the order of 1/2 inch may occur within all structures founded on compacted granular fill or till after construction.

Because the plant is founded on overconsolidated till, bedrock, or granular fill, no significant settlement will be caused by dynamic loads.

The No. 2 Coal (Colchester Member) was encountered in all the borings drilled to sufficient depth to penetrate it (57 borings), indicating the absence of any mining activity in the plant area. There is therefore no possibility of collapse or subsidence due to mines.

A system of construction settlement monuments was established for the foundations of Category I structures during 1977 and 1979 as shown in Figure 2.5-263. These monuments were installed and monitored by the contractor for the purpose of construction control and settlement monitoring. Seven of these monuments (U, V, N, R4, Z, KK, and XX) have been monitored continuously from the beginning of construction in 1977 to August 1980. Many of the other original monuments were discontinued because of construction interferences, and some were replaced in February 1979 by new monuments at similar locations within the same building. Other new monuments were also added at this time. In August 1980, monitoring was halted because settlement was complete under approximately 95% of the plant static load with measurements within the accuracy of the surveying equipment and methods used. Time settlement plots for all the construction settlement monuments which are shown in Figure 2.5-263 are presented in Figures 2.5-264 through 2.5-281.

In September 1981, a new set of operational settlement monuments was established throughout the plant. The intent of monitoring these monuments was to provide additional data to show that plant settlement under full static load is complete. The new monuments were installed on two floor levels which will reduce errors introduced into survey circuits by eliminating excessive traveling between different building levels. The locations of the operational settlement monuments are shown in Figure 2.5-282. Time settlement plots for all the operational settlement monuments are presented in Figures 2.5-283 through 2.5-295.

Table 2.5-41 is a summary of the maximum measured differential settlements for all construction and operational monuments. Table 2.5-42 is a summary of projected maximum total and differential settlements for each Category I structure. These total and differential settlements have been calculated after reviewing the stabilized elevations. The stabilized elevations have been identified on the settlement plots. Some allowance has been made in the total settlement due to the small amount of building load that had not yet been placed. The operational phase monuments, installed in September 1981, show that their maximum settlement measured through April 1988 is generally less than or equal to -0.01 feet (-0.012 feet maximum) except for Unit 2 containment monuments. The Unit 2 containment monuments numbers 41, 18, 17, R4, Z1 and Z show an average settlement of approximately -0.017 feet. This settlement is believed to be a result of small increases in dead load over the monitoring period and construction activities.

The differential settlements given in Table 2.5-42 are all less than or equal to -0.03 feet. This is significantly less than 1/2-inch or more which was assumed in the design of the auxiliary building and fuel handling building. The only safety-related pipe or conduit that is not suspended is the essential service water pipeline. This pipeline travels beneath the heater bay pipe or conduit and enters the turbine room mat. Beneath the heater bay, it is encased in reinforced concrete and supported on till or rock. The point of maximum differential settlement occurs as the encased pipeline enters the turbine room mat. The pipeline is designed to take with adequate margin the 1/2-inch estimated differential settlement in this area.

It is concluded that all Category I structures have been designed to account for the maximum total and differential settlement.

The lake screen house is founded within a very stiff to hard glacial till of the Wedron Formation. The till is overconsolidated and has an ultimate bearing capacity of approximately 45,000 psf (Subsection 2.5.4.10.1.2). The approximate static bearing pressure for the screen house is 3,000 psf resulting in a factor of safety of 15. The estimated settlement of the screen house is less than 1/4 inch total and 1/8 to 1/4 inch differential (Subsection 2.5.4.10.2.2). Construction phase settlement monitoring was not performed for the lake screen house but has been included in the operational phase settlement monitoring.

Six operational phase settlement monuments have been installed in the lake screen house to provide data to show that settlement is complete. The location of the monuments (60 through 65) are shown on Figure 2.5-282. Time settlement plots for these monuments are shown on Figures 2.5-294 and 2.5-295. The results given in Table 2.5-41 show maximum settlement values less than or equal to -0.01 feet (+0.007 feet maximum). This movement is considered negligible and indicates that settlement has stabilized.

The operational phase settlement monitoring program has continued with measurements at least four times per year from September 1981 to April 1988. The commitment to continue the program until 6 months after operation of Unit 1 has been met.

2.5.4.10.1.2 Bearing Capacity

The ultimate bearing capacity of the foundation bedrock was evaluated on a conservative basis, in accordance with methods described in Stagg and Zienkiewicz (Reference 95). The strength of the foundation rock was evaluated by means of rock compression tests. Using the appropriate values in Table 2.5-3 and considering these values to be representative for rocks with an RQD (Rock Quality Designation) of 100%, a reduction factor was selected on the basis of the measured RQD values. This reduction factor was used to arrive at a modified value approximating the in situ strength of the rock mass. Considering the uniformity of the rock formations and the number of borings, a minimum representative bearing capacity of the rock mass in the plant area is considered to be on the order of 150,000 psf. Using the maximum static bearing pressure on rock of 10,000 psf, the factor of safety against foundation failure is 15.

The ultimate bearing capacity of the in situ till was evaluated by means of unconsolidated-undrained triaxial compression tests and unconfined compression tests performed on samples obtained with the Dames & Moore Type U sampler. Any disturbance caused during sampling and handling operations tends to cause the results of the strength tests to be conservative. On this basis, using the results outlined in Table 2.5-11, the ultimate bearing capacity of the till is on the order of 45,000 psf for structures founded on the till. Using the maximum static loading on till of 5000 psf, the factor of safety against foundation failure is 9.

The ultimate bearing capacity for compacted granular fill was evaluated in accordance with methods described in Terzaghi and Peck (Reference 96) using the following expression:

$$q_u \quad \gamma(D_f \; N_q + 0.4 \; BN_\gamma) \qquad (2.5\text{-}6)$$

where:

q_u = ultimate bearing capacity (psf);

D_f = minimum imbedment depth (ft);

B = minimum foundation plan dimention (ft);

γ = moist unit weight above water level, bouyant weight below water level (pcf); and

N_q, N_γ = Terzaghi's bearing capacity factors.

Conservative values for the above parameters based on the laboratory test data presented in Table 2.5-10 were used for analyses and result in the expression:

$$q_u = 2000\ D_f + 1000\ B\ (\text{pcf}) \qquad (2.5\text{-}7)$$

The previous analyses give a relatively high ultimate bearing capacity for the mat foundations. Therefore, settlement considerations govern the design. Based on the settlement criteria of Subsection 2.5.4.10.1.1, ultimate bearing capacity was limited to 20,000 psf.

2.5.4.10.1.3 Lateral Pressures

All plant substructures were designed to resist lateral earth and water pressure at all levels below elevation 600 feet. All mat foundations established below 600 feet were designed to resist hydrostatic uplift pressures.

Subsurface walls were designed to resist both the static and dynamic pressures resulting from the surrounding earth and water.

2.5.4.10.1.3.1 Static Lateral Pressure

The total static lateral pressure was obtained by combining soil and hydrostatic pressures. Static lateral earth pressure on the wall at a depth, h, below grade is given as:

$$P_s = K_o h_\gamma \qquad (2.5\text{-}8)$$

where:

P_s = static lateral soil pressure, psf/linear ft;

K_o = lateral earth pressure coefficient for granular soil compacted against unyielding rigid walls;

As discussed below, the coefficient of lateral earth pressure at rest is conservatively taken to be 0.88 for the compacted granular backfill around Category I structures.

h = depth below grade, ft; and

γ = soil unit weight, 122 pcf above the water table and 67.6 pcf, submerged unit weight, below the water table.

Hydrostatic pressures are calculated using the equation:

$$P_w = 62.4y \qquad (2.5\text{-}9)$$

where:

P_w = hydrostatic pressure, in psk/linear ft, and

y = depth below the design water elevation, (ft).

Figures 2.5-300 and 2.5-301 detail the lateral earth pressure plots versus depth for the auxiliary building and the lake screen house. Hydrostatic pressure on Category I structures are computed as detailed in Braidwood UFSAR Subsection 2.5.4.10.1.3.1. The total static lateral pressure was obtained by combining soil and hydrostatic pressures.

Sources of conservatism in our earth pressure calculations are listed below:

a. All Category I subsurface walls and foundations are designed for a uniform construction surcharge load of 1,000 psf applied at grade for the normal loading conditions. Under normal plant operation conditions this surcharge load is not present.

b. The coefficient of lateral earth pressure at rest, K, for compacted granular backfill behind rigid walls is a function of the angle of internal friction, ø For the recompacted sands at Braidwood Station, the ø angle is 34°. This is a conservative number for compacted sand. Therefore, the coefficient of lateral earth pressure at rest, K, for compacted granular backfill behind rigid walls is conservative.

c. The coefficient of lateral earth pressure at rest for a sand with an angle of internal friction (ø) of 34° is 0.44. To increase the coefficient of lateral earth pressure at rest for the effect of compacting the soil in lifts the 0.44 coefficient was increased by a factor of two to give a coefficient of 0.88.

2.5.4.10.1.3.2 Incremental Dynamic Lateral Pressure

The total incremental dynamic lateral pressure was obtained by combining incremental soil and incremental water pressures. The total incremental dynamic lateral pressure was added to the static lateral pressure to obtain the design lateral pressure.

The dynamic lateral earth pressure increment on the subsurface walls was obtained by methods similar to those developed by Mononobe (Reference 97) and Okabe (Reference 98) and modified by Seed and Whitman (Reference 99).

The equation used to obtain the increment of dynamic lateral earth force was:

$$\Delta P_{AE} = 1/2\gamma \ H^{2} \ \Delta K_{AE} \qquad (2.5\text{-}10)$$

where:

ΔP_{AE} = increment of dynamic lateral earth force in pounds/unit width of wall,

γ = unit weight of soil in pcf (the submerged unit weight was used below the water table);

H = height of the wall in feet, and

ΔK_{AE} = dynamic increment in earth pressure coefficient

Values of ΔK_{AE} are a function of horizontal acceleration. For granular backfill, ΔK_{AE} is approximately equal to 3/4 K_h (Reference 99).

$$\Delta K_{AE} = 3/4 \ K_{h} \qquad (2.5\text{-}11)$$

where:

K_h = horizontal earthquake ground acceleration divided by the acceleration of gravity, g.

The dynamic earth pressure was assumed to have an inverted triangular distribution, with the resultant acting at two-thirds the height of the wall above the base.

The dynamic water pressure increment below the water table was calculated using the Westergaard theory (Reference 100) as modified by Matuo and Ohara (Reference 101). The increase in the pressure on the walls at any depth, y, below the water table is given as:

$$\Delta P_{W} = 0.70 \ CK_{h} \ (H_{1}y)^{1/2} \qquad (2.5\text{-}12)$$

where:

ΔP_W = water pressure, in pounds per feet per unit width of wall,

$K_h = \dfrac{\text{horizontal earthquake ground acceleration}}{g}$

$C = \dfrac{51}{[1.0 - 0.72\,(H_1/1000t)^2]^{1/2}}$

t = earthquake period (sec),

H_1 = height of the water table from the base of the wall (ft), and

y = depth below the water table (ft).

2.5.4.10.2 Pond Screen House

The retaining walls adjoining the pond screen house are reinforced concrete wing walls founded on Wedron silty clay till between elevations 561 feet 9 inches and 569 feet 0 inch. The walls are designed as Category I and extend as much as 100 feet east and west of the screen house. Plans and sections of the walls are given in Figures 2.5-317, 2.5-318, and 2.5-319.

2.5.4.10.2.1 Settlement

The pond screen house is supported on mat foundations established on the Wedron till. The base of the foundation is at elevation 565 feet 2 inches.

The compressibility characteristics of the foundation subgrade materials (till) were evaluated from the soil and rock properties summarized in Table 2.5-26 and the method of analysis described in Subsection 2.5.4.10.1.1. Since the soil and rock conditions at the pond screen house are essentially the same as at the plant site, the compressibility characteristics are similar and applicable. The total settlement of the structure due to static loads is less than 1/4 inch. Differential settlements will be on the order of 1/8 to 1/4 inch. Results of the settlement monitoring program for the pond screen house are presented in Subsection 2.5.4.10.1.1.

Dynamic settlement and possible differential settlement during the safe shutdown earthquake will be negligible for the glacial till and bedrock supporting the pond screen house.

2.5.4.10.2.2 Bearing Capacity

The ultimate bearing capacity of the foundation was evaluated as described in Subsection 2.5.4.10.1.2. Using the maximum static loading of 3000 psf for structures founded on the till the factor of safety against foundation failure is 15.

2.5.4.10.2.3 Lateral Pressures

The subsurface walls were designed as described in Subsection 2.5.4.10.1.3.

2.5.4.10.3 Essential Service Water Line and Discharge Structure

The essential service water discharge structure is founded on approximately 23 feet of Wedron glacial till deposit overlying the Carbondale bedrock formation. (See Boring H-4, Figure 2.5-159, Sheet 16.) The glacial till is stiff to hard and not susceptible to liquefaction.

The essential service water discharge structure is backfilled with previously excavated sand compacted to minimum 85% relative density in accordance with ASTM D-2049. Results of eight field density tests indicate relative densities ranging from 90% to 121% and averaging 104%. The average field dry density is 113.4 lb/ft^3. This backfill is not susceptible to liquefaction. The liquefaction potential of the adjacent natural sand deposit forming the essential service cooling pond (ESCP) foundation soils is discussed in Subsection 2.5.6.5.2.

A plan showing the location of the essential service water discharge structure with reference to the ESCP is given in Figure 2.4-28. Sections through the structure are given in Figure 2.5-302. The ESCP slope south of the structure is 10 horizontal to 1 vertical. The stability of the slope has been analyzed and results presented in Subsection 2.5.6.5.1.2.

In the event that a flow-type failure occurred as a result of the SSE, the discharge pipes would not be blocked with material from the slope. The invert of the discharge pipes is at elevation 591.0 feet. The top of the 10 to 1 slope is greater than 110 feet south of the discharge pipes and has been graded to elevation 590.0 feet. The toe of the interior dike is approximately 215 feet south of the discharge pipes at its closest point. The interior dike is of sufficient distance away from the discharge pipes to have no potential effect on their operation. It is concluded that the discharge pipes will not become blocked from any flow-type slope failure.

The factors considered in the static stability check of the essential service water discharge structure included the pipe discharge force, wave forces, weight of the structure, water pressure, buoyant forces, and seismic forces. The loading combinations incorporated SSE, OBE, and static loads and used two lake level elevations (598 feet 2 inches and 587 feet 0 inch). Elevation 598 feet 2 inches is the flood condition, and elevation 587 feet 0 inch is the low water condition that will occur if the lake dikes are damaged. Refer to Figure 2.5-302 for structural details.

The discharge structure has been checked for sliding, overturning, and bearing on the soil. Sliding is counteracted by the passive soil pressure developed along the sides of the structure and friction along the bottom of the structure. The overturning moments from seismic, wave, and discharge forces are offset by a resisting moment due to the deadweight of the structure. The bearing forces on the soil have been compared to the bearing capacity of the glacial till beneath the structure. The factors of safety are in accordance with those required by SRP Section 3.8.5.

The maximum bearing pressures and factors of safety against sliding and overturning modes of failure for both static and dynamic loading conditions are given in Table 2.5-47.

The essential service water lines are founded on till or bedrock as shown in Figure 2.5-25. Since the pipes exert no added load on the foundation materials, no settlement or bearing capacity computations were needed.

Since 10 borings taken along the pipeline route revealed the Colchester coal had not been mined there, no differential settlement due to subsidence from abandoned mines is anticipated.

2.5.4.11 Design Criteria

The criteria and methods used in the design of Seismic Category I structures are discussed in the following subsections:

a. bearing capacity, Subsection 2.5.4.10.1.2,

b. settlement analyses, Subsection 2.5.4.10.1.1,

c. slope stability, Subsection 2.5.6.5, and

d. lateral pressures, Subsection 2.5.4.10.1.3.

2.5.4.12 Techniques to Improve Subsurface Conditions

All Seismic Category I structures except a portion of the fuel handling building are founded on till or bedrock. Under the portion of the fuel handling building and adjacent non-Seismic Category I structures founded above the till, the inplace lacustrine sands were excavated and recompacted under controlled conditions to a minimum density of 85%, as discussed in Subsection 2.5.4.2.2. The effectiveness of the recompaction was verified by measuring inplace density of the fill and comparing with laboratory tests. This testing was supplemented with continuous observation of the placement and compaction operations by a soil engineer.

The testing was done by an independent testing laboratory (Pittsburgh Testing Laboratory).

The static stability of the recompacted sands is discussed in Subsection 2.5.4.10, and their liquefaction potential in Subsection 2.5.4.8.

2.5.4.13 Subsurface Instrumentation

A system of settlement benchmarks was established on the foundations of the plant structures as shown in Figure 2.5-89. The system was incorporated into the construction phase settlement monitoring program with settlement monuments located as shown in Figure 2.5-263. Discussion of this program and presentation of results is included in Subsection 2.5.4.10.1.1.

2.5.4.14 Construction Notes

No unanticipated conditions were encountered during the construction of the plant that required any change in the design or special construction techniques.

2.5.5 Stability of Slopes

The only Seismic Category I slopes are in the ultimate heat sink. The stability of these slopes is discussed in Subsection 2.5.6.

2.5.6 Embankments and Dams

2.5.6.1 General

This subsection presents the geotechnical design and the static and dynamic stability of the essential service cooling pond (ESCP).

The ESCP is located as shown in Figure 2.5-90 and serves as the ultimate heat sink for the plant. The ESCP is 6 feet deep, with the bottom at elevation 584 feet MSL. The side slopes of the pond are 10:1 horizontal-to-vertical. The pond is rectangular in shape and covers an area of 99 acres. The cooling water intake 1 structure is located in the northwestern corner of the ESCP. Discharge facilities for the essential service water are located at the southern end of the ESCP.

The ESCP is a Seismic Category I structure. In the unlikely event of a failure of the main perimeter dike and loss of all lake water above elevation 590.0 feet, the ESCP would be required to: 1) maintain its structural integrity to provide for circulation of the water supply, and 2) return an adequate supply of water for the safe shutdown of the plant. Detailed studies were made to evaluate the design with respect to two specific geotechnical safety considerations: (1) seepage, and (2) ground stability in terms of slope stability and liquefaction during the postulated SSE. The studies included: geotechnical investigation of the ESCP area; evaluation of the results of these investigations to determine site and subsurface conditions; and evaluation of the two specific geotechnical safety consideration using appropriate analytical methods and results of geotechnical investigations. Subsections 2.5.6.2, 2.5.6.5, and 2.5.6.6 present the results and conclusions of these studies.

2.5.6.2 Exploration

2.5.6.2.1 Purpose and General Scope

Geotechnical investigations were made to obtain information about the classification and distribution, the pertinent static properties, and the pertinent dynamic properties of soil and bedrock within and near the ESCF.

In general, the geotechnical investigation consisted of field and laboratory investigations. The scope of each is given in the following paragraphs.

2.5.6.2.2 Field Investigations

Field investigations consisted of drilling 22 borings (H-1 through H-4 and HS-1 through HS-18), excavating 7 test pits (HTP-1 through HTP-7), installing of piezometers in selected borings, performing water pressure tests (pump-in permeability tests) at various depths in bedrock in borings H-1 through H-4, and making 71 inferred determinations of the in-place relative density from standard penetration test data and 45 measured determinations of the in-place relative density of soil deposits below elevation 590.0. Figure 2.5-91 shows the locations of borings and test pits.

Logs of the borings are given in Figures 2.5-159 and 2.5-236 through 2.5-253. These logs show the depth of each boring, the classification of soil and bedrock penetrated, the location and type of samples obtained, the locations of piezometers, and other pertinent field test information such as standard penetration resistance. Logs of test pits are given in Figures 2.5-254 through 2.5-260 and show the depth of the test pit, sample locations, classification of soils encountered, and results of pertinent field and laboratory tests.

Results of water pressure tests in borings are summarized in Tables 2.5-20 through 2.5-23. Results of water level readings in piezometers are given in Table 2.5-25. Results of density determinations are summarized in Tables 2.5-30 and 2.5-31.

2.5.6.2.3 Laboratory Investigations

Laboratory investigations consisted of static and dynamic testing of selected samples obtained from borings and test pits. Static tests included index, strength, and permeability determinations made in accordance with applicable ASTM standards. Results of permeability tests are summarized in Table 2.5-24. Results of other static tests are summarized in Tables 2.5-32, 2.5-33, and 2.5-34. Dynamic tests included shear modulus, damping ratio, and cyclic strength determinations made in accordance with generally accepted procedures. Results of modulus and damping tests are given in Figures 2.5-77 through 2.5-80. Results of cyclic strength tests are summarized in Tables 2.5-35 through 2.5-38.

2.5.6.2.4 Evaluation of Exploration Results

Results of geotechnical investigations were evaluated to determine surface and subsurface conditions in the ESCP area.

2.5.6.2.4.1 Surface Conditions

The ESCP is located south of the main plant as shown in Figure 2.5-16, Sheet 1. The original ground surface is gently rolling from elevation 590 to elevation 600. During construction, ground surface was excavated to elevation 590 within the pond area, and to elevation 584 within the ESCP area.

The eastern boundary of the ESCP is located approximately 500 feet west and parallels an exposed existing strip mine excavation face (see Figure 2.5-90). The top of the strip mine excavation face was about elevation 600, and the slopes in some cases were near vertical. During construction, these excavations were backfilled to elevation 585 (see Figure 2.5-92). The fill consists primarily of soils from the pond excavations.

2.5.6.2.4.2 Subsurface Conditions

2.5.6.2.4.2.1 General

Evaluation of subsurface conditions in the ESCP area includes: 1) classification and distribution of soil and rock; 2) a determination of the static properties of major soil and rock deposits; and 3) a determination of the dynamic properties of the major soil deposits within the completed ESCP.

2.5.6.2.4.2.2 Classification and Distribution of Major Soil Deposits and Bedrock

Figure 2.5-92 shows generalized geologic sections that illustrate soil and bedrock conditions in the ESCP. The ESCP is underlain by three major soil deposits, which in turn are underlain by bedrock. The major soil deposits (in order of occurrence from the ground surface) are an organic topsoil and cohesive loess deposit; a lacustrine sand deposit; and a glacial till deposit.

The bedrock is a relatively flat-lying shale with occasional seams within the investigated zone. Site geology is discussed in Subsection 2.5.1.2.

2.5.6.2.4.2.3 Topsoil and Cohesive Loess Deposit

This deposit consists of a surface organic topsoil underlain by a variable thickness of a moderately cohesive loess deposit. The total thickness of the deposit is 3 to 4 feet. The topsoil portion was generally found to be 1 to 2 feet thick, and the cohesive loess portion generally approximately the same thickness. Since excavation within the pond to elevation 590.0 will result in removal of this deposit, no further evaluation of its distribution or properties is given.

2.5.6.2.4.2.4 Equality Formation Sand Deposit

The sand deposit is of lacustrine origin and consists of a dense to very dense fine sand, with varying amounts of silt and occasional silt nodules. The sand deposit contains two distinct strata clearly differentiated by color. Above approximately elevation 585, the sand deposit is typically brown to buff. Below elevation 585, the sand deposit is typically gray. The color difference is a result of weathering. Generally, the brown to buff sand contains the largest percentage of silt and silt nodules. Thin-section analyses were made of intact specimens taken from block samples. The analyses show the brown portion to be lightly to moderately cemented with calcium carbonate and iron oxide. Quantitatively, the analyses show the brown portion to contain 4% to 8% (by area) cement. The cement was formed during and in some respects by the weathering process. Generally, the thin-section analysis shows that the gray sand contains only traces of silt and silt nodules and is essentially uncemented.

The sand deposit below elevation 590 is typically 15 feet thick with the maximum thickness of 17 feet.

2.5.6.2.4.2.5 Wedron Glacial Till Deposit

The glacial till deposit consists of a very stiff to hard clayey silt, with occasional and intermittent lenses of very dense granular sands and fine gravels.

The glacial till has a typical thickness of 20 to 25 feet.

2.5.6.2.4.2.6 Bedrock

Upper bedrock units consist of interbedded shales and siltstones, with thin layers of sandstone, coal, and underclay of Pennsylvanian age. These units are variable in areal extent and thickness. Below these upper units are shale and dolomite strata of the Ordovician (Maquoketa) age. None of the borings or probes showed the bedrock to contain voids or other indications of significant solution or mining activity.

The top of bedrock was encountered from elevation 556.3 to elevation 552.0 with the average being elevation 552.9.

2.5.6.2.5 Static Properties of Major Soil Deposits and Bedrock

The following subsections present discussion of static properties of the two strata of sand deposits and the glacial till deposit considered most pertinent for evaluation of the geotechnical safety of the ESCP.

2.5.6.2.5.1 Equality Formation Sand Deposit

The two strata of sand consist of an upper brown silty fine sand and a lower gray medium to fine sand with trace of silt. Typical gradations for both strata are shown in Figure 2.5-94. Figure 2.5-95 shows the variation of fines content (percent minus No. 200 US standard sieve with opening of 0.074 mm) versus elevation for the sand deposit. The brown silty fine sand has an average fines content of 18.6% with an average D_{50} (50% of the sample is smaller than this diameter) of 0.16mm, and the gray fine sand has an average fines content of 5%, with an average D_{50} of 0.24mm.

Laboratory constant head permeability test results show the permeability of the sand deposit to range from 3.66×10^{-4} cm/sec. to 7.32×10^{-2} cm/sec. For the seepage evaluation, a value of 6×10^{-3} cm/sec was used.

Relative densities (D_r) were inferred from standard penetration test results (N data) and are plotted versus elevation in Figures 2.5-121 and 2.5-122. D_r was determined using N data, effective stress, and relationships developed by Gibbs and Holtz (Reference 102). Average values were calculated by assigning a maximum D_r of 95% for values plotting to the right of the curve relating N, σ and D_r =95% In the brown silty fine sand, the inferred D_r ranges between 62% and 95%, with an average of 85%. In the gray fine sand, the inferred D_r ranges between 60% and 95%, with an average of 87%.

Relative densities were also calculated based on results of field density tests and on laboratory vibratory compaction tests on 6-inch-diameter stratified samples obtained immediately adjacent to each field density test location. In the brown silty fine sand between elevations 590.0 and 585.0, the calculated D_r ranges between 51% and 100%, with an average of 80%. In the gray fine sand elevation 585.0, the calculated D_r ranges between 74% and 100%, with an average of 87%.

In two test pits (HTP-4 and HTP-5), the gray fine sand was encountered at about elevation 585.0. Relative density measurements ranged from 60% to 66%, with the average of three determinations being 62%. Borings HS-15 and HS-14 made adjacent to these test pits show consistently high N values and inferred D_r and do not confirm these generally lower measured D_r values.

On this basis, it is concluded that the somewhat lower measured D_r data obtained in gray fine sand found above elevation 585.0 are indicative of small, isolated and discontinuous medium dense to dense lenses on the surface of the deposit which should not be considered in overall evaluation of liquefaction potential of either the brown or gray portions of the sand deposit. The liquefaction potential of these isolated lenses is evaluated separately in Subsection 2.5.6.5.2.3.1.3.

Figure 2.5-96 shows results of maximum and minimum dry density tests on stratified samples of the sand deposit obtained adjacent to density test locations plotted versus fines content. It is seen that as fines content increases towards approximately 15%, the maximum and minimum density test results also tend to increase. For fines content greater than 15%, the density test results show no tendency to increase and in some cases show a tendency to decrease. For samples with greater than 12% fines content, the maximum density was also determined on remolded soils by the modified Proctor compaction procedure (ASTM D-1557). Results show that the vibratory compaction procedure (ASTM D-2049) gave consistently larger maximum dry unit weights than the modified compaction procedure. However, the maximum dry unit weight selected to calculate D_r was the larger of the two values. The minimum dry unit weight was determined using the soil from the stratified sample in general accordance with ASTM D-2049. Gradation of the sample was determined after each density determination to assure no degradation of the sample by the compactive effort.

It is concluded that the inferred and measured D_r values are in reasonable agreement with each other and indicate that overall the sand deposit is consistently dense to very dense.

2.5.6.2.5.2 Wedron Glacial Till Deposit

The average permeability of the glacial till was found to be 2.6×10^{-6} cm/sec. For well-graded fine gravel and silts at depths of 3.6 to 40.5 feet, the permeability was as high as 8.9×10^{-4} cm/sec.

2.5.6.2.5.3 Bedrock

The results of the pressure tests conducted in bedrock are presented in Tables 2.5-20 through 2.5-23 and on boring logs presented in Figures 2.5-159 and 2.5-236 through 2.5-253. The results indicate that the rock strata in the upper portions of the boreholes are more permeable than in deeper units. Permeabilities of up to 1.0×10^{-4} cm/sec are recorded in zones in the upper parts of these holes. These values are relatively low but are generally higher than those in deeper portions of the holes. These more permeable zones range in thickness from 10 feet in Boring H-4 to about 70 feet in Boring H-3. Below a maximum depth of 122 feet (Boring H-3), the permeability decreases and becomes very low to low, the highest being 1.1×10^{-5} cm/sec.

2.5.6.2.6 Dynamic Properties of Major Soil Deposits

Results of dynamic triaxial tests made to determine the modulus and damping properties of the sand deposit and glacial till deposit are given in Figures 2.5-77 through 2.5-80. Curves drawn through the data points were used for the geotechnical safety evaluation of the ESCP, which is discussed separately in Subsection 2.5.6.5.

Figure 2.5-99 shows the laboratory cyclic shear strength of the sand deposit, based on results of initial cyclic shear strength tests on reconstituted test specimens. Neither the relative density nor the fines content of test specimens was determined. Consequently, the initial test results were not considered in determination of the cyclic strength of either portion of the intact sand deposit. A comprehensive field and laboratory testing program, as discussed in Subsections 2.5.6.2.2 and 2.5.6.2.3, was designed and implemented to determine the cyclic shear strength curves for the intact sand deposit to be used in the geotechnical safety evaluation. The resulting laboratory cyclic shear strength curves are shown in Figure 2.5-100 and 2.5-101. A discussion of the test program implemented and analyses of the data obtained in determination of these curves is given in Subsection 2.5.6.5.2.2.1. Use of these laboratory curves in the liquefaction potential evaluation is discussed in Subsection 2.5.6.5.2.2.2.

2.5.6.3 Foundation and Abutment Treatment

2.5.6.4 Embankments

Refer to Subsection 2.5.6.2. This is not applicable to this site because the heat sink is excavated.

2.5.6.5 Slope Stability

The overall stability under static and dynamic conditions of the 10:1 horizontal-to-vertical side slopes of the ESCP was evaluated. In addition, the liquefaction potential of the sand deposit was also evaluated.

2.5.6.5.1 Slope Stability: Methods of Analysis

The static stability of the ESCP slopes was analyzed using the computer program SLOPE. See Appendix D for the program description. The factor of safety against sliding failure is computed using the theory of limiting equilibrium. The Bishop method of analysis was used for the ESCP slopes. The factors of safety were computed for various trial circular slope surfaces to find the most critical slip surface.

In the Bishop method of analysis, the factor of safety is defined as a ratio of the available shear strength of the soil to that required to maintain equilibrium under a postulated incipient failure. The failure is postulated along a circular arc surface, with plane strain conditions assumed to exist. The soil mass bounded by the face of the slope and the trial failure arch is divided into a finite number of slices. The equilibrium of each slice is considered separately. The resultant of all the forces on sides of the slice is assumed to act horizontally. The sum of these forces is neglected. The equilibrium of the slice in the vertical direction is satisfied to determine the unknown forces. The factor of safety is calculated in terms of moments of all the forces acting on slices around the center of the failure arc. The sensitivity of the factor of safety to the assumptions made in this simplified method is quite insignificant.

The overall stability of the slopes of the ESCP was calculated using the pseudostatic method of analysis. The earthquake loading on the slope was represented by a constant horizontal seismic coefficient applied to each slice. A seismic coefficient of 0.2g was used for the SSE and 0.1g for the OBE.

An analysis of the dynamic stability of the ESCP slope by finite element methods was not performed because the pseudostatic analysis used yields conservative results and a greater minimum factor of safety would be obtained if a finite element method were used. This is the case because the method of analysis employed assumes application of the seismic force at the base of each slice rather than at the centroid. It should be noted that factor of safety is determined by a comparison of overturning moments and resisting moments and that no consideration is given to the effects of side forces on slices in making computations. The seismic force is assumed to increase only the overturning moment and to have no influence on the resisting moment. The soil strength properties have also been based on triaxial compression tests rather than plane strain tests. This is also conservative. Discussion of the conservative nature of these assumptions can be found in the paper by H. B. Seed, K. L. Lee, and I. M. Idriss on the Analysis of Sheffield Dam Failure, Journal of the Soil Mechanics and Foundations Division, November 1969.

The minimum factor of safety for slope stability using pseudostatic analysis with a seismic coefficient of 0.2g was 1.3. With a seismic coefficient of 0.26g the minimum factor of safety is 1.1, which is considered acceptable.

2.5.6.5.1.1 Geometry, Loading Conditions, and Soil Properties

Figure 2.5-97 shows the geometry and soil properties used in the analysis. The geometry and soil properties were selected on the basis of geotechnical investigations to conservatively represent field conditions.

The following criteria for minimum factor of safety under various loading conditions were required for static analyses:

	Minimum Factor of Safety Required
a. End of construction case: no water	1.3
b. Submerged case: pool elevation at elevation 595.0	1.3
c. Rapid drawdown: pool elevation reduced from elevation 595.0 to elevation 590.0	1.4

The following criteria were required for pseudostatic analysis:

	Minimum Factor of Safety Required
a. Normal operating condition: pool elevation 595.0, SSE loading based on 0.2g seismic coefficient	1.1
b. Rapid drawdown; ESCP reduced from elevation 595.0 to elevation 590.0, SSE loading based on 0.2g seismic coefficient	1.1

	Minimum Factor of Safety Required
c. ESCP elevation 590.0, OBE loading based on 0.10g seismic coefficient	1.1

2.5.6.5.1.2 Results of Slope Stability Analyses

The critical section of the ESCP slope analyzed for static stability is given in Figure 2.5-313. The analysis is for end of construction condition.

The following summarizes the minimum factors of safety provided for the various loading conditions:

Loading Conditions	Minimum Factor of Safety Provided
a. Static Loading Conditions	
1. End of construction: no water	5.9
2. Normal operating conditions: ESCP water elevation 595.0	7.0
3. Rapid drawdown: ESCP water reduced from elevation 595.0 to elevation 590.0	7.0
b. Pseudostatic Loading Conditions	
1. Normal operating conditions: pool elevation 595.0, SSE loading based on 0.2g seismic coefficient	1.3

2.	Rapid drawdown: ESCP water drop from elevation 595 to elevation 590.0, SSE loading based on 0.2 seismic coefficient	1.3
3.	ESCP elevation 590.0, OBE loading based on 0.10g seismic coefficient	2.3

The ESCP slope has also been analyzed with a seismic coefficient of 0.26g and the minimum factor of safety is 1.1.

2.5.6.5.2 Liquefaction Potential

The liquefaction potential of the sand deposit forming the ESCP slopes and foundation soils was evaluated using the results of the geotechnical investigation and a detailed dynamic response analysis.

2.5.6.5.2.1 Method of Analysis

The cyclic shear stresses likely to be induced by the postulated safe shutdown earthquake (SSE) are calculated by computing the shear stress time history for discrete layers of the subsurface profile. The first part of the procedure involves the development of a design accelerogram such that its free field response spectrum closely matches the Regulatory Guide 1.60 response spectrum. Such an accelerogram was obtained as described in Subsection 3.7.1.

Next, the nonlinear strain-dependent, dynamic strength, and compressibility properties of the subsurface deposits are established from laboratory cyclic test results and field investigations. Then the computer program SHAKE (described in Appendix D) is used to compute the soil response to SSE loading.

The induced irregular shear stress time-histories are computed at various depths within the soil profile.

By appropriate weighting of the stress levels involved in various stress cycles throughout the postulated earthquake and by introducing the soil cyclic shear strength curve data, the irregular shear stress time-history is converted into an equivalent time-history of uniform stress levels. In this manner, all the significant factors influencing liquefaction stability, including: 1) intensity of ground shaking, 2) duration of ground shaking, and 3) strain-dependent nonlinear soil parameters, are taken into account in the analysis.

Finally, these induced shear stresses corresponding to the significant number of cycles for the postulated SSE and the shear stress required to cause liquefaction are compared for various depths to evaluate the liquefaction potential of the sand deposit.

2.5.6.5.2.2 Geometry, Soil Properties, and Earthquake Time-History

Figure 2.5-98 shows the geometry and the static soil properties used for the analyses. Figures 2.5-77 through 2.5-80 show the modulus and damping properties used for the sand and glacial till deposit for the analyses. Figures 2.5-100 and 2.5-101 show the laboratory cyclic shear strength curves for the two portions of the sand deposit.

The synthetic earthquake time-history used in the analysis is given in Figure 2.5-102.

2.5.6.5.2.2.1 Cyclic Shear Strength of Equality Formation Sand Deposit

A comprehensive field and laboratory testing program was developed and implemented to determine the design cyclic shear strength of the intact sand deposit. The following method was used in the determination:

a. Determine by means of borings, test pits and laboratory tests the classification and distribution of the sand deposit throughout the ESCP.

b. Establish by indirect and direct tests the representative relative density of the intact sand deposit.

c. Determine the design cyclic strength representative of the intact sand deposit by making laboratory tests on either intact or reconstituted samples at relative densities equal to those determined in a. and b. above.

2.5.6.5.2.2.2 Laboratory Tests to Determine Cyclic Strength

The classification and distribution of the sand deposit are presented in Subsection 2.5.6.2.4.2.4. Of significance to the establishment of the cyclic strength of the intact sand deposit are the fines content and the relative density of the two portions of the deposit.

Two fines content values were selected for each portion of the sand deposit based on statistical evaluation of results of the fines content test data. The first represents the average; the second represents a "low average" value established as the fines content at which 67% of test results were greater. Figure 2.5-103 shows a plot of the cumulative percent of occurrences of the fines content test results. Using the criteria established the following fines contents were selected:

Portion	Average Fines Content (%)	Low Average Fines Content Values (%)
Brown silty fine sand between elevation 590.0 and elevation 585.0	19	13
Gray medium to fine sand below elevation 585.0	5	2

Both indirect and direct tests show the sand deposit to be consistently dense to very dense. Direct tests are considered more applicable and showed that the average D_r of the deposit above elevation 585.0 was slightly less than the average D_r of the deposit below elevation 585.0.

Figure 2.5-104 shows a plot of the cumulative percentage of occurrences of the measured D_r results for the brown and gray portions of the sand deposit. Using the statistical criteria established to select fines content, the following D_r values were selected.

Portions	Average D_r Value (%)	Low Average D_r Value (%)
Brown silty fine sand between elevation 590.0 and elevation 585.0	80	77
Gray medium to fine sand below elevation 585.0	87	80

A two-phase laboratory testing program was designed and implemented to determine the cyclic shear strength of the two portions of the intact sand deposit. Phase 1 consisted of performing stress-controlled, cyclic triaxial tests on reconstituted test specimens to evaluate cyclic strength as a function of fines content and relative density. Phase 2 consisted of performing stress-controlled, cyclic triaxial tests on a series of "companion" intact and reconstituted test specimens of approximately the same relative density to obtain data to evaluate the effect of the change in the soil structure, caused by reconstituting the soil specimens, on the cyclic strength. The scope and results of the test programs follow.

Phase 1

Three series of tests were made using reconstituted test specimens. Test Series 1 consisted of 11 tests made using specimens formed of soil with a fines content of approximately 1%. Results were considered representative of the low fines-content portion of the gray fine sand deposit below elevation 585.0. Test Series 2 consisted of eight tests made using specimens formed of soil with a fines content of about 11%. Results were considered representative of lower fines content portions of the brown silty fine sand deposit above elevation 585.0. Test Series 3 consisted of five tests made using specimens formed of soil with a fines content about 20%. Results were considered representative of the average fines-content portion of brown silty fine sand deposit above elevation 585.0. D_r values of the test specimens in each series were varied to provide a representative range of D_r as indicated by direct test results. Results of Phase 1 test are given in Tables 2.5-35 and 2.5-36.

As can be seen from Figures 2.5-100 and 2.5-101 and the data tabulated in Tables 2.5-35 through 2.5-38, the number of cycles to cause failure of samples ± 10% strain is much greater than that to cause ± 5% strain. This trend is consistent with the dense nature of the sand deposit.

Figure 2.5-105 shows the effect of fines content and D on cyclic strength measured using reconstituted specimens. The strength is shown as the stress ratio to cause ± 5% strain in 10 cycles. Analyses of these data indicate that the cyclic strength increases as the fines content and relative density increase. Similar variations were observed for other definitions of cyclic strength.

A comparison was made between laboratory cyclic strength curves determined for the sand deposit and comparable laboratory cyclic strength curves for two other clean fine sands for which extensive data are available, the Monterey sand and the Sacramento River sand.

The comparison was made for ± 10% axial strain considering tests on reconstituted test specimens formed using a "wet tamping" compaction procedure to a D_r of 80%. Results are shown in Figure 2.5-106. The curve for Monterey sand was extrapolated from tests made at D_r = 60%. The extrapolation was made assuming a linear relationship between D_r and (d /2$\Phi_{,c}$ for the D_r range of 60% to 80%. The curve for Sacramento River sand was determined by adjusting a curve for D_r = 80% determined using reconstituted test specimens formed using a "wet raining" compaction procedure. Recent data (Reference 103) indicate the laboratory cyclic strength of sand is increased by approximately 30% if test specimens are formed using the "wet tamping" compacting procedure rather than the "wet raining" procedure. On this basis, laboratory curves for Sacramento River sand at D_r = 80& were adjusted by multiplying published ratios by 1.30.

Analysis of Figure 2.5-106 indicated that the curve for the relatively clean gray sand compares very well. The curve for the relatively high fines-content brown silty fine sand is about 25% greater than the curve for low fines-content clean sand. Several similar comparisons made at different relative densities and axial strain levels indicated similar comparisons.

Phase 2

The correspondence of results of cyclic tests on reconstituted test specimens to the cyclic strength of the intact sand deposit was established by a series of tests. In this series, tests were made on specimens trimmed from intact block samples obtained from test pits. "Companion" reconstituted test specimens were formed on the trimmed block sample material and reconstituted to provide a D_r for the reconstituted test specimens approximately equal to D_r of the intact specimens. Results of Phase 2 tests are given in Tables 2.5-37 and 2.5-38.

Figure 2.5-107 shows typical results of tests on an intact specimen and a "companion" reconstituted specimen adjusted to approximately equal D_r. It is seen that for strain levels below some strain level, ε_c, (referred to a crossover strain) results of reconstituted specimens are "stronger" (exhibit lower strain response to the same number of stress cycles) than results on intact specimens. Consequently, for strains less than ε_c, use of test results on reconstituted specimens would yield unconservative estimates of cyclic strength. The degree of unconservatism is a function of the strain difference (strain difference is equal to the difference between ε_c and the strain level selected to define liquefaction) and the relative density of the deposit. Generally, the amount of unconservatism decreases as the strain difference decreases and the relative density increases. Conversely, for strains greater than ε_c, use of test results on reconstituted specimens would yield somewhat conservative estimates of cyclic strength. The data obtained show that for D_r greater than 70%, the use of cyclic strength determined using reconstituted samples is only slightly unconservative at strain levels less than 5% double amplitude and is very conservative at other levels.

Figure 2.5-108 shows ε_c plotted as a function of D_r. The trend of these data indicate that ε_c decreases with increasing Dr and that for Dr values greater than about 75%, ε_c is less than ± 2% axial strain. The minimum "low average" D_r for the sand deposit is greater than 75%. This would indicate that ε_c for reconstituted specimens would be less than ± 2% and that the strength interpreted from the tests on reconstituted test specimens would be less than the intact strength of the sand deposit. It is concluded that use of strength interpreted from tests on reconstituted samples would give a conservative estimate of the cyclic strength of the intact sand deposit.

2.5.6.5.2.2.3 Use of Laboratory Test Data to Determine Cyclic Strength of Intact Equality Formation Sand Deposit

The field cyclic strength of the sand deposit is determined by the adjustment of laboratory cyclic strength curves developed from results of cyclic triaxial tests on reconstituted soil specimens (Figures 2.5-100 and 2.5-101) to allow for (1) effect of specimen reconstitution on the fabric of the intact sand; and (2) differences in stress conditions between laboratory and field. The adjustments to accommodate the above effects are made using the following equation:

$$(\tau_f / \sigma_v)_f = D_c C_r \ (\sigma_d / 2\sigma_{3C})_L \qquad (2.5\text{-}13)$$

where:

$(\tau_f/\sigma_v)_f$ = stress ratio representative of field cyclic strength of intact sand deposit,

D_c = correction factor to adjust for effect of specimen reconstitution,

C_r = correction factor to adjust for effect of differences in stress conditions between field and laboratory, and

$(\sigma_d/2\sigma_{3C})_L$ = stress ratio representative of laboratory cyclic strength of reconstituted test specimens.

A quantitative estimate of the relationship between the cyclic strength of intact test specimens to the cyclic strength of "companion" reconstituted test specimens (D_c) (cyclic strength of Sacramento River sand was determined using "wet raining" compaction procedure) was made using both Phase 1 and Phase 2 test data and the procedure outlined in Figure 2.5-109. The results (D_c) are plotted versus D_r of "companion" Phase 2 test specimens in Figure 2.5-110 for initial liquefaction, ± 5% axial strain, ± 10% axial strain. The curves drawn in Figure 2.5-110 represent linear regression curves through the data points considered representative of the sand deposit. As indicated on the figure, data points were not considered representative when significant differences were measured between gradation characteristics of the companion specimens.

Analysis in the curves shown reveals two distinct trends in the variation of D_c. The first is that D_c increases as the magnitude of the axial strain to define liquefaction increases. The second is that D_c increases as D_r of the "companion" test specimens increases. The shaded area represents the range of D_r assigned to the sand deposit for the liquefaction potential evaluation. It is seen that D_c ranges from 0.98 to 1.43 for the range of D_r assigned to the sand deposit.

Two techniques were utilized to determine the factor C_r, which is a factor to adjust for differences in stress conditions. The first procedure has been widely used by geotechnical engineers in recent years and makes use of a relationship between C_r and D_r proposed by Seed and Peacock (Reference 105). The second procedure makes use of shaking table test (conducted at the

University of California at Berkeley) results made available by Dr. H. B. Seed in April 1975. These results indicate that C_r is essentially independent of relative density. Based on data developed by Seed and Peacock (Reference 105), however, C_r is related to K_o (Reference 105).

The selection of C_r for use at this site was therefore made first based on D_r and second by taking into account only the variation of K_o. The field strength curves for intact sand deposit developed by both procedures were used in the liquefaction potential evaluation and the results compared.

2.5.6.5.2.2.4 Selection of C_r Based on Relative Density

The curve of C_r versus D_r given in Seed and Peacock (Reference 105) is presented in Figure 2.5-111. The relationships were developed by tests on typical, clean fine sands such as the Sacramento River sand and the Monterey Sand. To determine D_r to be used to select C_r, an equivalent evaluation of the laboratory cyclic strength was made. The equivalent evaluation consisted of comparison of laboratory cyclic strength of the sand deposit with laboratory cyclic strength of the Sacramento River sand. The evaluation is shown in Figure 2.5-112. It is seen that for 14 cycles or less, the sand deposit within the FSCP exhibits a laboratory cyclic strength greater than that for the Sacramento River sand reconstituted to a D_r of 90%. On this basis, an equivalent D_r of 90% was used to select a value for C_r of 0.75.

2.5.6.5.2.2.5 Selection of C_r Based on K_o

The data from shaking table liquefaction test results, including the effect of two directions shaking together with data presented by Seed and Peacock (Reference 104), indicate that for $K_o = 0.4$, $C_r=0.57$, and for $K_o=1.0$, $C_r=0.9$. A linear interpretation between these points has been suggested by Dr. H. B. Seed of the University of California at Berkeley, as presented in Figure 2.5-113. The increase in Ko as developed due to the construction of the ESCP is shown in Figure 2.5-114. The curve shown in Figure 2.5-115 (Reference 106) relates K_o to overconsolidation ratio (OCR) for sands typical of sand deposits within the ESCP. The K_o values within the completed ESCP, presented with respect to elevation, are shown on Figure 2.5-114. From these K_o values and the proposed relationship between C_r and K_o, C_r values can be determined as a function of elevation in the soil profile. Values of C_r vary from 0.65 at elevation 570.0 to 0.83 at elevation 585.0. Soils above elevation 585.0 were assumed to have a C_r of 0.83.

The cyclic strength of the intact sand deposit is determined by adjusting the curves in Figures 2.5-100 and 2.5-101 using the equation given previously. In fact, the curves are adjusted by multiplying by a factor $C_r \times D_c$ where the value of C_r varies with the method of selection.

2.5.6.5.2.3 Evaluation of Liquefaction Potential

2.5.6.5.2.3.1 Stresses to Cause Liquefaction

An earth structure such as the ESCP can undergo deformation on the order of a few feet without being considered to have failed. For this reason and based on the results of detailed evaluation of the San Fernando Dams (Reference 107), a failure criterion of ± 10% axial strain was selected to evaluate the liquefaction potential of the ESCP. In addition, the potential behavior using lower strain criterion (initial liquefaction and ±5% strain) was investigated.

The data presented in Figures 2.5-100 and 2.5-101 together with the applicable effective overburden pressure and the correction factor, C_r x D_c were used to calculate the stress required to cause liquefaction in 10 cycles at selection depth within the soil profile. Thus, at a depth, y (below elevation 590.0), the following equation was used to calculate the stress to cause liquefaction, τf:

$$\tau_{f_{10}} = C_r \cdot D_C \cdot \sigma_{v(y)} \cdot (\sigma_d / 2\sigma_{3C})_{L_{10}} \qquad (2.5\text{-}14)$$

where:

$\tau_{f_{10}}$ = shear stress to cause liquefaction of intact sand 10 deposit in 10 cycles,

D_C = correction factor to adjust for effect of specimen reconstitution,

C_r = correction factor dependent on D or K as appropriate,

$\sigma_{v(y)}$ = effective overburden pressure at depth y, and

$(\sigma_d / 2\sigma_{3C})_{L_{10}}$ = stress ratio representative of laboratory cyclic strength of reconstituted test specimens at 10 cycles.

Using the above equation, the resulting distribution of shear stress to cause ± 10% axial strain with depth is presented in Figures 2.5-116 and 2.5-117 for C_r based on D_r and C_r based on K_o, respectively.

2.5.6.5.2.3.1.1 Induced Stresses

As discussed in Subsection 2.5.6.5.2.1, the stresses induced by the SSE throughout the depth of the soil profile were computed using the program SHAKE. The time-history of stress at each depth was converted to a uniform shear stress and 10 cycles. The resulting distribution of stresses, τ_d, induced in 10 cycles by the SSE is shown in Figures 2.5-116 and 2.5-117.

2.5.6.5.2.3.1.2 Determination of τ_f/τ_d

The induced stresses, τ_d, and the stresses required to cause ±0% strain (τ_f) are compared in Figures 2.5-116 and 2.5-117. The ratio τ_f/τ_d (which represents a factor of safety) is shown plotted with depth in the soil profile in Figures 2.5-116 and 2.5-117. The maximum and minimum τ_f/τ_d ratios for the brown silty fine sand deposit and the gray fine sand deposit are tabulated in Tables 2.5-39 and 2.5-40 for C_r based on D_r and C_r based on K_o, respectively. Values of τ_f/τ_d are presented for "average" and "low average" conditions corresponding to the development of initial liquefaction, ±5% axial strain, and ± 10% axial strain.

In summary, for "average" soil conditions and C_r based on D_r, the minimum factor of safety for the development of ± 10% strain and initial liquefaction is 2.3 and 1.2, respectively.

For comparison, if C_r is based on K_o, the minimum factor of safety for development of ± 10% strain and initial liquefaction is 2.6 and 1.3, respectively.

2.5.6.5.2.3.1.3 Effect of Lenses of Medium Dense Gray Fine Sand in the Wedron Till Above Elevation 585.0

The field investigation has indicated that small, discontinuous lenses of medium dense to dense (D_r as low as 60%) gray fine sand occur at the site. The cyclic strength of these lenses would be less than that used in the liquefaction evaluation. Using C_r as a function of D_r, the factor of safety for these lenses against failure (± 10% strain) would be approximately 1.0 and would be less than 1.0 for initial liquefaction. Using C_r as a function of K_o, the factor of safety for these lenses for the development of ± 10% average strain or initial liquefaction would be greater than 1.25. Using the most conservative of the results, it is concluded that the SSE would cause, at most, small settlements of these localized lenses of material. Thus the presence of these lenses within or near the slope would not impair the integrity of the slope.

2.5.6.5.2.3.1.4 Effect of Evaluation at Elevation 584.0 Feet

The liquefaction potential of the ESCP bottom at the Braidwood Station was also evaluated at elevation 584.0 feet. Factors of safety against liquefaction are calculated and presented for level ground at elevations 590 feet and 584 feet, and "average" and "low average" relative density conditions corresponding to the development of initial liquefaction (IL), ±5% axial strain, and ±10% axial strain.

Calculations indicating the various correction factors and the resulting factors of safety are presented in Tables 2.5-49 through 2.5-52.

The induced stresses, (τ_d), and the stresses required to cause ±10% strain (τ_f) for level ground at elevation 590 feet are compared in Figures 2.5-116 and 2.5-117, and for level ground at elevation 584 feet in Figures 2.5-315 and 2.5-316. Figures 2.5-315 and 2.5-316 are plots of data from Table 2.5-51 which correspond to average relative density conditions.

Selection of Parameters

The following parameters were used to calculate τ_f/τ_d:

Parameter	Description and Source
τ_d	Shear stress induced by SSE. τ_d values are plotted with depth of the soil profile in Figures 2.5-116 and 2.5-117. These were computed based on a SHAKE analysis for level ground at elevation 590 feet. τ_d values used in the calculations for level ground at elevation 584 feet, were calculated by using a simplified procedure for evaluating stresses described in Reference 118 (Seed & Idriss, 1982). The SHAKE program has been run for level ground at elevation 584 feet and verifies that the τ_d values shown in Tables 2.5-51 and 2.5-52 are conservative.
$(\sigma_d/2\sigma_{3C})$	Stress ratio representative of laboratory cyclic strength of reconstituted test specimens at N=10 stress cycles. $(\sigma_d/2\sigma_{3C})$values vs. N are plotted in Figures 2.5-100 and 2.5-101 for soil type and relative density/fines content properties.
D_c	Correction factor to adjust for effect of specimen reconstitution. D_c values are plotted in Figure 2.5-110 and are dependent on relative density and strain condition.
C_r	Correction factor dependent on relative density or K_o, as appropriate. The selection of C_r based on D_r was made on the basis of an equivalent Sacramento River Sand D_r = 90% and the curve presented in Figure 2.5-111. The C_r value for D_r = 90% is 0.75. The selection of C_r based on K_o was obtained from Figure 2.5-113. The value of K_o was obtained based on OCR from Figure 2.5-115. The OCR was calculated as shown in Figure 2.5-314 for level ground at elevation 584. For OCR greater than 4.5 a K_o value of 0.88 is selected. For K_o = 0.88, a value of C_r = 0.83 is selected from Figure 2.5-113.

Calculation Method

The calculated factors of safety (FS) are presented in Tables 2.5-49 through 2.5-52. Three FS values are calculated (columns

(8), (11), and (14) for each elevation and strain condition considered. The method used to calculate FS is as follows:

(1) FS in column (8)

$$\frac{(FS) = C_r \cdot \sigma_v \ \sigma_d / 2\ 3C)}{\tau_d}$$

where $C_r = 0.75$

(2) FS (C_r based on D_r) in column (11)

FS (C_r based on D_r) = FS $\cdot$ D_c

(3) FS (C_r based on K_o) in column (14)

FS (C_r based on K_o) on column (14)

FS (C_r based on Ko) = Fs (Cr based on Dr) $\cdot \frac{C_r}{0.75}$

where C_r obtained from Figure 2.5-113.

2.5.6.5.2.3.2 Conclusion

Based on results of the liquefaction potential evaluation, it is concluded there is an ample margin of safety against liquefaction of the sand deposits within the ESCP for level ground surfaces at both elevations 590 feet and 584 feet. It is further concluded that the ESCP will maintain its structural integrity and provide for continuous circulation of the essential water for safe shutdown of the plant if necessary during the unlikely event of the postulated SSE.

2.5.6.5.3 Analysis of the Interior Dike Located West of the ESCP

Cross-section details of the interior dike located west of the ESCP are given as Section 18 in Figure 2.4-35. Plan view of the interior dike west of the ESCP is given in Figure 2.4-28. The toe of the interior dike at elevation 590 feet is located approximately 80 feet west of the top of the ESCP slope.

The static and dynamic stability analyses for the interior dike are summarized in Table 2.5-53.

The interior dike is not a Category I structure and was not designed for SSE loading. It has been designed to be stable under OBE loading.

The effect of failure of the interior dike on the ESCP was investigated by conservatively assuming that the entire failure slip circle of soil is deposited downstream beginning at the interior dike toe. This surcharge of failed soil will remain 50 feet or more away from the top of the ESCP slope and thereby not act as a critical surcharge at the head of the ESCP slope.

In the unlikely event that material from a failed portion of the interior dike did enter the ESCP, the volume of soil is so small that it would have an insignificant effect on the operation of the ESCP.

2.5.6.6 Seepage Control

2.5.6.6.1 Methods of Analyses

Seepage studies for the ESCP were carried out using finite element techniques such as described by R. L. Taylor (Reference 108). These studies were made using the computer program SEEPAGE (see Appendix D).

2.5.6.6.2 Analysis Conditions

Input for the computer program included the pond and site geometry, boundary conditions, soil till interface, directional permeabilities for the soil in flow domain, maximum available differential head, and initial trial phreatic surface. The trial phreatic surface line is corrected through an iterative procedure to satisfy the flow conditions.

The analysis was made with the assumption that under an unlikely SSE event, the cooling pond is completely drained and a 10-foot differential head is caused between the ESCP water surface and the groundwater table. (This is supported by data from LW-2 for the period from July 1973 to May 1977 which indicate a median groundwater level of 580.1 feet.) An average value of coefficient of permeability equal to 2×10^{-4} ft/sec (6.0×10^{-3} cm/sec) was assigned to sand deposits in both the horizontal and vertical directions. The permeability for till was found very small compared to sand, and till was therefore input as impervious boundary. The flow was assumed to occur primarily in an easterly and southerly direction. Flow to the north and west is negligible because of the slurry cutoff trench beneath the perimeter dike of the cooling lake. A plan view showing the boundary of the cooling lake which coincides with the perimeter dike slurry trench is given in Figure 2.4-37.

(The slurry trenches shown in Figure 2.5-74 are trenches that were installed prior to main plant and lake screenhouse construction to assist in construction dewatering of these facilities. These trenches are not considered permanent installations and should not be confused with the cooling lake perimeter dike slurry trench.)

The cooling lake perimeter dike slurry trench is continuous around the perimeter of the cooling lake and, therefore, continuous along the north and west sides of the essential service cooling pond (ESCP). Plan views of the ESCP and perimeter dike along the north and west sides of the ESCP are given in Figures 2.4-26 through 2.4-29. Sections of the perimeter dike and slurry trench are given in Figure 2.4-35. The slurry trench along the

ESCP is a soil-bentonite backfilled slurry trench extending from elevation 597 feet to top of till and, in most cases, are keyed into till. As-built profiles are provided in Figures 2.5-310, 2.5-311, and 2.5-312.

The design of the ESCP does not rely on the slurry trench as a seepage barrier. The ESCP seepage has been conservatively determined assuming the slurry trench does not exist. Existance of the slurry trench only makes the seepage analysis more conservative.

2.5.6.6.2.1 Aquifer Description

The Equality Formation is composed primarily of fine to medium grained sands with some silt layers. This is based on a review of all the soil samples at the project site. Review of the essential service cooling pond borings, HS-1 through HS-18, and H-1 through H-4, and available grain size analyses indicates that the Equality Formation beneath the ESCP consists of dense to very dense, silty fine sands (SM) to fine sand (SP and/or SP-SM). Figures 2.5-84, 2.5-94, and 2.5-118 present grain size curves for these soils.

In some borings and mapped sections, a 1 to 4 foot thick layer of coarse gravel, cobbles, and boulders in a fine to medium grained sand matrix occurs directly above the clay till of the Wedron Formation. The thickness of the gravel and cobbles is not 1 to 4 feet, but is contained within a 1 to 4 foot thick layer of fine sand, silty sand, and/or clayey sand. For instance, in geologic Section 23, there are two 4 to 6 inch layers of fine to coarse gravel near the bottom with 1 to 2 inches of lag gravel at the bottom.

Based on the review of logs of borings drilled in or near the ESCP, no borings reveal a 1 to 4 foot thick layer of coarse gravel, cobbles, or boulders within the Equality Formation. Only boring HS-14 indicated the presence of a thin gravel layer approximately 0.4 foot thick directly above the clay till layer. Boring H-1 located 800 feet east of the ESCP indicated a lag gravel layer within the Wedron till formation beneath a 3.5 foot thick layer or silty clay till.

Six additional borings DSS-1 and DSS-66 through DSS-70 were drilled near the ESCP. Their locations are shown in Figure 2.5-90. Logs of these borings are shown in Figures 2.5-303 through 2.5-308. All of these borings show the lag gravel within the Wedron Formation and in most cases within a silty clay matrix.

In summary, a review of the ESCP borings conclusively shows that a layer of coarse gravels, cobbles, and/or boulders does not exist in the Equality Formation beneath the ESCP and in most cases, the gravel that is encountered is found within the Wedron Formation in a silty clay matrix or beneath a layer of silty clay till.

2.5.6.6.2.2 Aquifer Thickness

The seepage analyses for the ESCP are based on an aquifer thickness of 13 feet below elevation 584 feet. Based on the review of HS-series, DSS-1 and DSS-66 through DSS-70 borings, the thickness of the aquifer below elevation 584 feet in the ESCP area, in general, varies from 0.3 feet to 11.5 feet with few exceptions. Borings DSS-69 indicates a thickness of 13.1 feet and boring HS-5 indicates a thickness of 16.5 feet. The average thickness of the sand layer is approximately 9 feet. See Figure 2.5-93 for profiles within the ESCP.

Therefore, the seepage analysis based on a 13-foot thick layer of aquifer is very conservative with respect to the actual average of in situ conditions.

2.5.6.6.2.3 Coefficient of Permeability (k) Values

It was reported in an earlier response that "the k values of SP and SM material ranged from 7.37 x 10^{-2} cm/sec to 3.658 x 10^{-4} cm/sec with an average permeability of approximately 6.7 x 10^{-3} cm/sec/." These values are based on all the available laboratory data on permeability. However, if the soil samples obtained from the borings drilled in the ESCP area only are considered, which is more representative, the k values for SP and/or SM materials range from 10.0 x 10^{-3} cm/sec to 8.0 x 10^{-4} cm/sec and averages 4.3 x 10^{-3} cm/sec. These permeability values for the ESCP area are given in Table 2.5-43.

The laboratory permeability values for the HS series borings are constant head permeability tests on relatively undisturbed samples. The samples were obtained using an Osterberg sampler and the tube was fitted to a permeameter.

The laboratory k values were compared with k values estimated from the grain size distribution of selected soil samples. Permeability values were estimated based on: (a) D_{10} size using the Allen Hazens formula; (b) D_{10} size and uniformity coefficient C_u; and, (c) D_{20} size using the United States Bureau of Soil Conservations (USBSC) formula. The first two emprical relations are based on the laboratory test results. The Braidwood laboratory test results are within the range of these estimated values. The third empirical relation, USBSC's formula, yields the best correlation with coefficients of permeability based on pumping tests. The estimated k values based on this empirical relationship are lower (i.e., impervious) than that of the Braidwood laboratory test results. Comparison k values discussed here are reported in Table 2.5-43.

The seepage analysis reported in Subsection 2.5.6.6.2 used an average value of coefficient of permeability equal to 6 x 10^{-3} cm/sec which is conservative with respect to the average value obtained for the ESCP from laboratory samples.

2.5.6.6.2.4 Boundary Conditions

In our seepage analysis the downstream exit point is approximately 380 feet from the bottom edge of the ESCP, which approximately corresponds to the nearest mine spoil. The hydraulic head with respect to the water level at the exit point is 6 feet based on boring logs drilled before construction of the cooling lake. However, a 10-foot head was used in the analysis. This head and the shortest exit point have yielded a higher gradient than actually existed resulting in higher quantity of seepage.

The lowest water level recorded in piezometer LW-2, close to the ESCP, is at elevation 577.5 feet. The hydraulic head in the ESCP with respect to this lowest water level is 12.5 feet. However, the shortest distance between the bottom edge of the ESCP and the piezometer LW-2 is approximately 4,700 feet. This corresponds to a much lower hydraulic gradient than that of the one used in our analysis. Therefore, the seepage analysis reported in the FSAR is on the conservative side.

The north and west sides of the ESCP are located close to the perimeter dike of the cooling lake. See Figures 2.4-26 through 2.4-29. The perimeter dike has a slurry trench installed from elevation 597 feet to the top of the Wedron till and in most cases keyed into till. The trench is a soil-bentonite backfilled trench and will significantly reduce the amount of seepage from the ESCP even if the cooling lake dike fails. The seepage analysis assumes this slurry trench does not exist and is therefore conservative.

2.5.6.6.3 Results of Seepage Analyses

Pressure test results indicate that minor seepage occurs down through jointing in the shale bedrock. The possibility of large water loss from the ESCP as a result of solution cavities or mined-out areas is considered improbable for the following reasons:

a. No solution cavities or mined-out areas have been encountered in any boreholes made within the pond area or neighboring plant area. The rock underlying the cooling pond consists of interbedded siltstones, shales, coals, and a local channel of sandstone. In general, the permeability of these rocks is relatively low. They are not subject to solution activity.

   Below these rocks is the Fort Atkinson Limestone at a depth of about 125 feet. This thick limestone is porous, as shown by vugs and occasional porous zones noted on the logs. The water pressure tests show, however, very low permeabilities for this formation, indicating that the rock is relatively impermeable and that solution phenomena, if present, are minor.

b. Figure 2.5-36 indicates that the area occupied by the ESCP which is just south of the plant site does not contain any mined-out areas. This drawing represents the results of an exhaustive records search as described in Subsection 2.5.1.2.7.2. Additional borings, labelled DSS in Figure 2.5-90, were made for the pond perimeter dike. These borings penetrated the bedrock below the coal seams. These borings show no evidence of mined-out areas around the outer perimeter of the pond in the heat sink area. Subsection 2.5.1.2.1.2 also indicates that borings made by the Peabody Coal Company on 330-foot centers have not revealed any mined-out areas in the N 1/4 of Section 19, T32N, R9E. This area contains the plant and ESCP.

During a 30-day period, 1 foot of water would evaporate. Results of seepage study indicate that over a 30-day period the level of the pond would drop 0.5 foot. The combined water loss during a 30-day period due to seepage and evaporation would lower the surface of the ESCP by 1.5 feet. The design depth of the ESCP is 6.0 feet.

The seepage analysis is conservative for the following reasons:

a. Aquifer thickness used in analysis is 4 feet greater than the average thickness determined from the ESCP borings.

b. The gradient used in the analysis is higher than the actual gradient that existed before lake filling. In the event of a cooling lake failure, the ground at elevation 590 will remain saturated for some time. Any hydraulic gradient which may be established away from the ESCP will develop very slowly due to gravity drainage of the fine sands. Since seepage through these sands has been cut off by a slurry trench installed around the entire perimeter of the cooling lake, there should be no hydraulic gradient established. Photographs of the ESCP have documented that after the slurry trench was installed the ESCP remained under groundwater before the cooling lake was filled. The seepage analysis performed assumed no presence of a slurry trench cutoff and instantaneous development of a hydraulic gradient greater than anticipated or measured at any location prior to construction of the cooling lake. These factors are extremely conservative.

c. The coefficient of permeability used in the analysis is representative of the sands of the Equality Formation. It has been shown that it compares well with correlations using grain size and correlations developed using pumping tests.

An additional seepage analysis has also been performed using more conservative coefficients of permeability. The coefficients of permeability in the vertical direction and horizontal direction are 6.7×10^{-3} cm/sec and 2.0×10^{-2} cm/sec, respectively. The differential head used is again 10 feet which has been shown to be very conservative. Again the seepage was determined assuming the slurry trench does not exist. Results indicate that the drop in elevation of the ESCP due to seepage over a 30-day period is approximately 1.5 feet. This seepage drop combined with loss due to evaporation of approximately 1 foot gives a total drop of 2.5 feet or to elevation 587.5 feet. The pond surface area at elevation 687.5 is 95.4 acres based on an October 1981 hyrographic survey of the ESCP. The 95.4 acres is equal to or greater than the design area at elevation 590 feet as shown in Figure 9.2-8.

The analysis and discussions above clearly show that the seepage analysis is conservative and that the ESCP has ample supply of cooling water for the 30-day period.

2.5.6.7 Diversion and Closure

This subject is not applicable to the Braidwood site.

2.5.6.8 Instrumentation

An extensive cooling lake monitoring program has been followed since the beginning of lake filling on December 1, 1980. The ESCP is a submerged pond within the cooling lake and has been monitored as part of the overall cooling lake program and the Braidwood Stations' Surveillance Requirements. A complete summary of the cooling lake monitoring, including field observations and results of instrumentation are presented in the following report:

> Report GD-9 Braidwood Lake Monitoring Program, Six Year Monitoring Summary Report, December 1980 to December 1986, dated January 1987

The report contains monitoring data consisting of the dike settlement measurements, dike and mine spoil slope indicator movements, observation well water level and quality measurements, lake peripheral drainage, river stage readings, and aerial and hydrographic surveys. The hydrographic surveys were bottom and slope contours of the ESCP. The survey consists of making a recording of depth measurements at specific time intervals along track lines, spaced equally over the pond. Also, included in this report are the results of the surveys in terms of the surface area and volume capacity.

Monitoring of the ESCP is covered by Surveillance Requirement 4.7.5.

2.5.6.9 Construction Notes

The ESCP is an excavated pond within the cooling lake. Design and in situ soil conditions were presented in subsections above. The ESCP does not depend upon man-made structural features for water retention and is constructed to remain intact during a design basis seismic event.

2.5.6.10 Operational Notes

Field observations and results of instrumentation for the ESCP are discussed in Subsection 2.5.6.8.

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2.5.8 Individual and Agencies Contacted

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2. Grundy County, Office of the Recorder, Morris, Illinois.

3. Illinois Archives, Springfield, Illinois.

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9. Kentucky Geological Survey: E. M. Wilson.

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TABLE 2.5-1

SUMMARY OF MAJOR FOLDS WITHIN 200 MILES OF THE SITE

NAME	IDENTIFICATION*	MAJOR MOVEMENT**
Ashton Arch	B	Late Paleozoic (Ref. 20)
Baraboo Syncline	S	Precambrian* (Ref. 110)
Brookville Dome	B	Late Paleozoic (Ref. 15)
Clay City Anticline	B	Late Paleozoic (Ref. 114)
Downs Anticline	B	Late Paleozoic (Ref. 114)
DuQuoin Monocline	U	Post-Mississippian, Middle Pennsylvanian (Ref. 17)
Fond du Lac Syncline	B	Late Paleozoic
Forreston Dome	S, B	Late Paleozoic (Ref. 15)
Herscher Dome	B	Late Paleozoic (Ref. 20)
Illinois Basin	S, B, G	Early to late Paleozoic (Ref. 4)
Kankakee Arch	S, B, G	Ordovician or Devonian to late Mississippian (Ref. 4)
LaSalle Anticlinal Belt	S, B, G	Late Mississippian and Pennsylvanian (Ref. 4)
Leesville Anticline	S, Str	Late Mississippian, Early Pennsylvanian (Ref. 18)
Leaf River Anticline	S, B	Late Paleozoic (Ref. 15)
Lincoln Anticline	S	Late Paleozoic (Ref. 115)
Louden Anticline	B	Late Paleozoic (Ref. 114)
Marshall Syncline	Sc	Post-Mississippian, Pre-Mesozoic (Ref. 10)
Mattoon Anticline	B	Late Paleozoic (Ref. 114)

TABLE 2.5-1 (Cont'd)

NAME	IDENTIFICATION*	MAJOR MOVEMENT**
Meekers Grove Anticline	B	Late Paleozoic (Ref. 28)
Michigan Basin	S, B, G	Middle Ordovician to Middle Pennsylvanian, Jurassic (Ref. 19)
Mineral Point Anticline	B	Late Paleozoic (Ref. 28)
Mississippi River Arch	S, B, G	Lake Mississippian (Ref. 40)
Murdock Syncline	U	Post-Mississippian Pre-Mesozoic (Ref. 14)
Oregon Anticline	B	Late Paleozoic (Ref. 20)
Pittsfield Anticline	B	Late Paleozoic (Ref. 114)
Polo Basin	B	Late Paleozoic (Ref. 20)
Salem Anticline	U	Post-Mississippian, Pre-Mesozoic (Ref. 109)
Tuscola Anticline	U	Post-Mississippian, Pre-Mesozoic (Ref. 14)
Uptons Cave Syncline	S, B	Late Paleozoic (Ref. 15)
Waterloo Syncline	B	Late Paleozoic
Wisconsin Arch	S, B, G	Early to late Paleozoic (Ref. 4)

Notes:

Structures listed in this table are shown on Figures 2.5-9, 2.5-10 and 2.5-9a.

* S = Surface mapping
B = Borehole
G = Geophysical
Str = Structure
U = Undifferentiated

** Final movement considered to be pre-Cretaceous except as noted.

TABLE 2.5-2

SUMMARY OF FAULTS WITHIN 200 MILES OF THE SITE

NAME	IDENTIFICATION***	FAULT TYPE AND DISPLACEMENT	LAST MOVEMENT
(Appleton)*, ** (inferred)	S, B (Ref. 29)	South side down	Post-Silurian, pre-Pleistocene (Ref. 30)
Cap Au Gres Faulted Flexure	S, B	1000 ft of structural relief (Ref. 26)	Post-Mississippian (Ref. 26)
Centralia	S, B	Down 200 ft on west side (Ref. 17)	Post-Pennsylvanian (Ref. 17)
Chicago Area Basement Fault Zone	G (Ref. 23)	South side down	Precambrian (Ref. 23)
Chicago Area Minor Faults (inferred)	G (Ref. 22)	Both north and south downthrown blocks, displacement up to 55 ft (Ref. 22)	Post-Silurian, pre-Pleistocene (Ref. 22)
Des Plaines Disturbance	Sc (Ref. 42)	Radial and concentric approx. 5 mi. diameter	Post Pennsylvanian (Ref. 33)
Fortville	B (Ref. 111)	Southeast side down 60 ft (Ref. 111)	Post-Devonian, pre-Pleistocene (Ref. 111)
(Green Bay) (inferred)	S (Ref. 29)	South side down (Ref. 29)	Post-Silurian, pre-Pleistocene (Ref. 30)

Note: Structures listed in this table are shown in Figure 2.5-9.

* Name assigned by Dames & Moore.
** Most recent authorities doubt the existence of these faults (Ref. 24, 30)
*** S = surface, B = borehole, G = geophysical, Sc = structure.

TABLE 2.5-2 (Cont'd)

NAME	IDENTIFICATION***	FAULT TYPE AND DISPLACEMENT	LAST MOVEMENT
(Lanesville) (Ref. (inferred)	B (Ref. 29)	North side down (Ref. 29)	Post-Silurian, pre-Pleistocene (Ref. 30)
Mt. Carmel	S, B (Ref. 18)	West side down 80 to 175 ft (Ref. 18)	Early Pennsylvanian (Ref. 18)
(Madison)** (inferred)	B (Ref. 29)	North side down (Ref. 29)	Post-Silurian, pre-Pleistocene (Ref. 30)
Mifflin	S (Ref. 27, 28)	South side down 65 ft, 1000 ft strike-slip displacement (Ref. 27, 28)	Pre-Pleistocene (Ref. 30)
Northeast-trending faults north of Rough Creek Fault Zone (Wabash Valley Fault Zone)	S	Both east and west blocks downthrown (Ref. 39, 40)	Post-Pennsylvanian, Pre-Pleistocene (Ref.10)
Northeast-trending faults south of Rough Creek Fault Zone	S, B	Both east and west blocks downthrown (Ref. 39, 40)	Post-Pennsylvanian, Pre-Pleistocene (Ref. 10)
Oglesby* (inferred)	B (Ref. 12)	Down on west side 1200 ft (Ref. 12)	Pre-Cretaceous
Plum River Fault Zone	S, B, G (Ref. 15)	North side down 100 to 400 ft (Ref. 15)	Post-Silurian, pre-Pleistocene (Ref. 15)
Rough Creek Fault Zone	S (Ref. 34) B (Ref. 34) G (Ref. 113)	North side down (Ref. 39, 40) some members show opposite displacement	Post-Pennsylvanian, pre-Pleistocene (Ref. 10)

TABLE 2.5-2 (Cont'd)

NAME	IDENTIFICATION***	FAULT TYPE AND DISPLACEMENT	LAST MOVEMENT
Royal Center	B (Ref. 31)	Southeast side down 100 ft (Ref. 31)	Post-Devonian, pre-Pleistocene (Ref. 111)
Ste. Genevieve Fault Zone	S (Ref. 34) B (Ref. 34) G (Ref. 113)	North side down 1000 to 2000 ft (Ref. 10)	Post-Pennsylvanian, pre-Pleistocene (Ref. 5)
Sandwich Fault Zone	S, B, G	Main fault: northeast side down 900 ft (Ref. 116) Subsidiary fault: southeast side down 125 ft (Ref. 46)	Post-Pennsylvanian, pre-Mesozoic (Ref. 22)
Tuscola** (inferred)	B (Ref. 12)	Down on west side 2000 ft (Ref. 12)	Pre-Cretaceous
Waukesha** (inferred)	S (Ref. 112) B	Downthrown on southeast side 45 ft (Ref. 112) extent of fault is inferred (Ref. 30)	Post-Silurian, pre-Pleistocene (Ref. 30)

NOTES

1. Structures listed in this table are shown in Figure 2.5-9.

2. S = Surface, B = Borehole, G = Geophysical, Sc = Structure.

TABLE 2.5-3

UNCONFINED ROCK COMPRESSION TEST DATA, PLANT SITE BORINGS

BORING NO.	DEPTH (ft)	GEOLOGIC UNIT	ULTIMATE COMPREESIVE STRENGTH (psi)	ELEVATION (ft)
A-2	60	Francis Creek	7,286	533.0
A-2	95	Spoon	5,510	498.0
A-2	122	Brainard	7,142	471.0
A-2	125	Fort Atkinson	7,122	468.0
A-2	158	Fort Atkinson	1,878*	435.0
A-2	165	Scales	8,469	428.0
A-2	252	Galena	9,591	341.0
A-3	215	Scales	5,918	383.3
A-5	63	Francis Creek	6,735	535.5
A-5	101	Francis Creek	5,414	497.5
A-5	126	Fort Atkinson	3,121	472.5
A-5	146	Fort Atkinson	8,551	452.5
A-11	115	Spoon	8,600	487.0
A-11	142	Fort Atkinson	14,333	460.0
A-11	161	Fort Atkinson	12,611	441.0
A-11	163	Fort Atkinson	6,688	439.0
A-17	73	Francis Creek	980*	526.0
A-18	63	Francis Creek	3,469	538.4

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*Shear break

TABLE 2.5-3 (Cont'd)

BORING NO.	DEPTH (ft)	GEOLOGIC UNIT	ULTIMATE COMPRESSIVE STRENGTH (psi)	ELEVATION (ft)
MP-3	108	Spoon	5,270	492.2
MP-11	67	Francis Creek	7,090	532.8
MP-18	160	Fort Atkinson	10,900	438.9
MP-18	189	Scales	6,660	409.9
MP-25	73	Francis Creek	4,850	524.2
MP-30	175	Fort Atkinson	10,600	426.1
MP-32	79	Francis Creek	2,420	523.9
MP-38	121	Spoon	4,020	481.6
MP-45	124	Spoon	7,070	477.8
MP-50	41	Channel Sandstone	2,720	560.5

TABLE 2.5-4

RESONANT COLUMN TEST DATA

BORING NO.	ELEVATION (ft)	SOIL OR ROCK TYPE	FIELD MOISTURE CONTENT (%)	FIELD DRY DENSITY (lb/ft^3)	CONFINING PRESSURE (lb/ft^2)	SHEAR STRAIN AMPLITUDE (%)	SHEAR WAVE VELOCITY (ft/sec)	MODULUS OF RIGIDITY (lb/ft^2)	DAMPING (%)
A-1	563.9	ML	9.7	134	1,000	0.0080	568	$1.47x10^6$	3.7
		(Wedron			2,000	0.0058	697	$2.21x10^6$	4.3
		Formation)			3,000	0.0041	852	$3.30x10^6$	3.8
TP-1*	589.8	SM	11.9*	114*	1,300	0.0084	568	$1.28x10^6$	1.0
		(Equality			2,050	0.0058	719	$2.05x10^6$	1.2
		Formation)			2,800	0.0045	840	$2.78x10^6$	0.8
A-4	558.1	ML	10.7	130	1,000	0.0054	727	$2.35x10^6$	3.4
		(Residual			2,000	0.0051	752	$2.52x10^6$	3.7
		Soil)			3,000	0.0046	793	$2.80x10^6$	3.5
A-5*	578.5	SP	21.0	101	750	0.0096	522	$1.04x10^6$	0.8
		(Equality			1,500	0.0074	625	$1.49x10^6$	0.6
		Formation)			2,250	0.0062	700	$1.87x10^6$	0.5
A-6*	575.2	SP	17.8	109	750	0.0090	542	$1.17x10^6$	0.8
		(Equality			1,500	0.0069	650	$1.68x10^6$	0.6
		Formation)			2,250	0.0058	725	$2.09x10^6$	0.6
A-3	526.3	Siltstone	2.0	148	3,975	0.0015	2911	$39.77x10^6$	3.3
		(Francis			6,000	0.0015	2978	$41.63x10^6$	(avg)
		Creek)			8,000	0.0014	3018	$42.74x10^6$	

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*Test specimen was recompacted from bulk sample from test pit. Moisture content and dry density of test specimen differ from in situ values (see Table 2.5-13).

TABLE 2.5-4 (Cont'd)

BORING NO.	ELEVATION (ft)	SOIL OR ROCK TYPE	FIELD MOISTURE CONTENT (%)	FIELD DRY DENSITY (lb/ft^3)	CONFINING PRESSURE (lb/ft^2)	SHEAR STRAIN AMPLITUDE (%)	SHEAR WAVE VELOCITY (ft/sec)	MODULUS OF RIGIDITY (lb/ft^2)	DAMPING (%)
A-3	438.3	Limestone	2.0	165	10,000	0.0013	3353	58.67×10^6	
		(Fort			12,000	0.0014	3337	58.09×10^6	
		Atkinson)			14,000	0.0014	3343	58.32×10^6	
A-3	411.3	Limestone	2.0	167	10,000	0.0014	2938	45.67×10^6	
		(Fort			12,000	0.0014	2955	46.22×10^6	3.3
		Atkinson)			14,000	0.0014	2973	46.78×10^6	(avg)
MP-12	536.4	Siltstone	2.0	147	4,000	0.0014	2648	32.56×10^6	4.1
		(Francis			6,000	0.0012	2720	34.33×10^6	(avg)
		Creek)			8,000	0.0011	2738	34.79×10^6	
MP-18	455.8	Limestone	2.0	168	10,000	0.0008	1695	15.29×10^6	--
		(Fort			12,000	0.0013	2178	25.24×10^6	--
		Atkinson)			14,000	0.0009	4154	91.82×10^6	5.5
MP-29	570.5	ML	11.7	127	1,000	0.0169	575	1.46×10^6	5.9
		(Wedron			2,000	0.0197	569	1.43×10^6	6.6
		Formation)			3,000	0.0132	778	2.60×10^6	3.5
MP-32	505.0	Shale	2.0	148	5,000	0.0016	2389	26.69×10^6	2.7
		(Francis			7,000	0.0014	2337	25.52×10^6	(avg)
		Creek)			9,000	0.0015	2274	24.17×10^6	
MP-30	570.6	ML	11.7	128	1,000	0.0193	670	1.99×10^6	4.1
		(Wedron			2,000	0.0140	698	2.16×10^6	4.1
		Formation)			3,000	0.0156	766	2.56×10^6	3.7

TABLE 2.5-4 (Cont'd)

BORING NO.	ELEVATION (ft)	SOIL OR ROCK TYPE	FIELD MOISTURE CONTENT (%)	FIELD DRY DENSITY (lb/ft^3)	CONFINING PRESSURE (lb/ft^2)	SHEAR STRAIN AMPLITUDE (%)	SHEAR WAVE VELOCITY (ft/sec)	MODULUS OF RIGIDITY (lb/ft^2)	DAMPING (%)
MP-3	532.0	Siltstone	2.0	147	6,000	0.0028	3100	44.64×10^6	4.6
		(Francis			8,000	0.0271	3471	56.00×10^6	2.4
		Creek)			10,000	0.0388	3554	58.69×10^6	2.4
MP-17	456.1	Limestone	2.0	170	10,000	0.0012	3277	57.98×10^6	4.0
		(Fort			12,000	0.0011	3529	67.40×10^6	4.0
		Atkinson)			14,000	0.0011	3361	60.99×10^6	4.2

TABLE 2.5-5

CHARACTERISTICS AND PRODUCTION OF MINES IN THE AREA OF INTEREST

MAP NO.	COMPANY OR MINE	LOCATION SECTION - T.N. - R.E.	TYPE OF MINE	COAL SEAM NO.	THICKNESS	DEPTH	AREA (acres)	ACTIVE YEARS	TONS OF COAL PRODUCED
1	Eureka Coal Co., Mines No. 1 and No. 2	N1/2 18-32-9	U	2	3 ft	125 ft	37(1)	1872-1884	180,000(5)
2	Braidwood Coal Co.	SW1/4 17-32-9	U	2	u	u	u	Circa 1879	u
3	Cotton Mine	NW1/4 25-32-8	U	u	u	u	u	Circa 1883	u
4	Augustine Mine	NW1/4 25-32-8	U	u	u	u	u	Circa 1883	u
5a	Wilmington & Springfield Coal Co.	SE1/4 24-32-8	U	2	3 ft	107 ft	u	1872-1885(22)	
5b	Chicago, Wilmington & Vermillion Coal Co., "K" Mine	SE1/4 24-32-8	U	2	3 ft	107 ft	80	1885-1888	294,000(6) (23)
6	Chicago, Wilmington & Vermillion Coal Co., "M" Shaft	SW1/4 19-32-9	U	2	3 ft	95 ft	74(1)	1889-1891	277,845(5)
7	Braceville Coal Co., Mine No. 2	NW1/4 24-32-8(8)	U	2	3 ft, 6 in.	115 ft	10(2)	Abandoned 1894	43,300(6)
8	Chicago, Wilmington & Vermillion Coal Co., "R" Mine	NE1/4 13-32-8	U	2	3 ft	99 ft	56(1)	1896-1899	202,062(5)

TABLE 2.5-5 (Cont'd)

MAP NO.	COMPANY OR MINE	LOCATION SECTION - T.N. - R.E.	TYPE OF MINE	COAL SEAM NO.	THICKNESS	DEPTH	AREA (acres)	ACTIVE YEARS	TONS OF COAL PRODUCED
9	Braceville Coal, Co., Mine No. 4	SW1/4 13-32-8 and NW1/4 24-32-8	U	2	3 ft, 2 in.	103 ft	128(2)	1893-1900	520,000(6) (10)
10	Rixson Coal Co., Rixson No. 1 Mine	N1/4 30-32-9	U	2	3 ft, 2 in.	100 ft +	13	1903-1906	57,071(5)
11	Braceville Coal Co., Mine No. 5	NW1/4 18-32-8(8)	U	2	3 ft	103 ft	160(2)	1900-1908	620,000(6)(11)
12	Joliet-Wilmington Coal Co., Mine No. 2(21)	NW1/4 20-32-9	U	2	3 ft	115 ft	31	1905-1909	150,363(5)
13	Consolidated Coal & Iron Co.(8) (12)	SW1/4 19-31-9	U	3	3 ft	80 ft	32(2)	1905-1911	125,785(5)
14	Wilmington Coal Mining & Mfg. Co., No. 6 Mine (Torino)	NW1/4 31-32-9	U	2	3 ft	90 ft	197(9)	1905-1921	1,061,482(5) (13)
15	Truckers Coal Co.	N1/2 36-32-8	U	2	3 ft	115 ft	2(14)	1938-1940	1,221(5)
16	No. 3 Coal Corporation	W1/2 24-32-8(8)	U	3	u	u	148(2)	1927-1954	572,000(6) (15)
17	Gardner Wilmington Coal Co.	NW1/4 19-31-9	U	2+2A, 3	4 ft, 11 in. and 3 ft	106 ft and 80 ft	160(2)	1890-1904	1,344,391(5)
18	Unidentified Mine A	NE1/4 18-31-8	U	2	3 ft(16)	70 ft(16)	13(16)	u	43,000(6)

TABLE 2.5-5 (Cont'd)

MAP NO.	COMPANY OR MINE	LOCATION SECTION - T.N. - R.E.	TYPE OF MINE	COAL SEAM NO.	THICKNESS	DEPTH	AREA (acres)	ACTIVE YEARS	TONS OF COAL PRODUCED
19	Unidentified Mine B	NW1/4 18-31-8	U	2	3 ft	90 ft(16)	32(2)	u	124,000(6)
20	Unidentified Mine C	NE1/4 25-32-8	U	2	3 ft	u	153(2)	u	586,000(2)
21	Wilmington Coal Mining Co.	NE1/4 16-32-9	S	2	u	u	40(2)	Circa 1934	<154,000(6)
22	Wilmington Coal Mining Co.	Sections 21, 28, 29, 33-32-9	S	2	3 ft, 4 in.	60 to 80 ft	860	1940-1958	3,166,159(5) (17)
23	Wilmington Coal Mining Co.	NW1/4 17-32-9	S	2	u	u	32(3)	Circa 1940	138,000(6)
24	Northern Illinois Coal Co., Pit No. 11	Sections 5, 6, 7, 8 31-9	S	2	3 ft	40 to 90 ft	1300	1947-1956	5,000,000(6)
25	Peabody Coal Co. (Northern Mine), Pit No. 11	Sections 20, 21, 29, 30, 31, 32 32-9	S	2	3 ft	60 to 100 ft	1870	1958-1972	6,008,303(5)
26	Peabody Coal Co. (Northern Mine), Pit No. 12	12-31-8 and 7-31-9	S	4	3 ft, 8 in.	50 ft ± (24)	591	Post-1955	2,732,000(6) (18)

TABLE 2.5-5 (Cont'd)

MAP NO.	COMPANY OR MINE	LOCATION SECTION - T.N. - R.E.	TYPE OF MINE	COAL SEAM NO.	THICKNESS	DEPTH	AREA (acres)	ACTIVE YEARS	TONS OF COAL PRODUCED
27	Peabody Coal Co. (Northern Mine), Pit No. 13	Sections 17 and 18 31-9	S	2	3 ft	50 to 60 ft	142	1955-1957	548,000(6)
28	Peabody Coal Co. (Northern Mine), Pit No. 14	13-31-8 and 18-31-9	S	4	3 ft, 8 in.	40 to 70 ft	176	1969-1972	910,000(6) (19)
29	Peabody Coal Co. (Northern Mine), Pit No. 15	19-31-9	S	2	3 ft	50 to 60 ft	288	Post-1969	1,110,000(6)
30	Peabody Coal Co. (Northern Mine), Pit No. 16	1-31-8	S	4	3 ft, 8 in.	50 ft ± (24)	173	Post-1969	823,000(6) (20)
					UNDERGROUND PRODUCTION SUBTOTAL				6,202,520
					STRIP PRODUCTION SUBTOTAL				20,539,462
					TOTAL				26,741,982

TABLE 2.5-5 (Cont'd)

NOTES

U	Underground mine
S	Strip mine
u	Unknown
(1)	Area estimated from production tonnage at 74% recovery.
(2)	Area estimated from mined-out area map (Figure 2.5-82).
(3)	Area estimated from aerial photograph (1971).
(4)	Area estimated from unpublished data in files of Dames & Moore.
(5)	Production tonnage reported in Illinois Coal Reports.
(6)	Production tonnage estimated from mined area at 74% recovery.
(7)	Production within area of interest only.
(8)	Access shaft or principal workings in adjoining section beyond area of interest.
(9)	Does not include shaft pillar.
(10)	Total production reported for entire mine in Sections 13, 14, 23, and 24 was 1,797,653 tons.
(11)	Total production reported for entire mine in Sections 13 and 14 was 1,051,399 tons.
(12)	Operated 1908-1911 by Clarke City Wilmington Coal Co.
(13)	Tonnage computed from mine map area is 1,060,000 tons.
(14)	Development workings only.
(15)	Area of underground workings indicates production history prior to 1927. Operated by Skinner Coal Co., 1927-1937; by the South Wilmington Coal Co., 1938-1943; no production, 1944-1947; and by No. 3 Coal Corporation, 1948-1954.
(16)	Thickness of coal, depth, and area determined from surrounding drill-hole data.
(17)	Production erroneously attributed to Section 17, T.32N., R.9E. in Illinois Coal Reports.
(18)	Total production from Pit 12 including Section 11 is estimated at 3,700,000 tons.
(19)	In Kankakee County, 190,000 tons; in Grundy County, 720,000 tons.
(20)	Production from entire Pit 16 is estimated at 3,700,000 tons of product.
(21)	In 1909, the company name was changed to Joliet & Aurora Coal Co.
(22)	Mine later extended as Chicago, Wilmington & Vermillion Coal Co. "K" Mine.
(23)	Tonnage includes that of previous mine workings.
(24)	From Illinois Coal Reports.

TABLE 2.5-6

TYPICAL COAL ANALYSES

SAMPLES		PROXIMATE					HEAT VALUES	
COUNTY, NUMBER OF MINES, COAL	CONDITIONS*	MOISTURE	VOLATILE MATTER	FIXED CARBON	ASH	SULFUR	CALORIES	Btu
Grundy	1	17.1	37.4	39.7	5.8	2.8	6139	11,050
Four mines**	2		45.1	47.9	7.0	3.3	7402	13,320
Colchester (No. 2) coal	3		48.5	51.5		3.5	7959	14,330
	4	18.6	38.8	42.6			6574	11,830
	5		47.7	52.3			8009	14,520
Grundy	1	13.8	38.7	38.3	9.3	3.54	6052	10,894
One mine	2		44.8	44.4	10.3	4.11	7020	12,636
No. 4 coal	3		50.2	49.7		4.60	7868	14,162
	4	15.6	41.5	42.9			6765	12,171
	5		49.2	50.8			8019	14,401
Will	1	15.4	34.2	45.3	5.1	1.6	6299	11,340
One mine**	2		40.5	53.5	6.0	1.9	7449	13,410
Colchester (No. 2) coal	3		43.1	56.9		2.1	7928	14,270
	4	16.5	35.4	48.1			6682	12,030
	5		42.4	57.6			7998	14,400
Kankakee	1	15.0	36.4	42.8	5.8	2.77	6394	11,510
One mine	2		42.8	50.4	6.8	3.25	7520	13,536
Colchester (No. 2) coal	3		45.9	54.1		3.49	8066	14,520
	4	16.3	37.8	45.9			6851	12,331
	5		45.1	54.9			8176	14,717

TABLE 2.5-6 (Cont'd)

SAMPLES		PROXIMATE					HEAT VALUES	
COUNTY, NUMBER OF MINES, COAL	CONDITIONS*	MOISTURE	VOLATILE MATTER	FIXED CARBON	ASH	SULFUR	CALORIES	Btu
Kankakee	1	14.4	38.0	38.8	8.9	3.42	6072	10,930
One mine	2		44.4	45.3	10.4	3.99	7093	12,767
No. 4 coal	3		49.5	50.5		4.45	7911	14,239
	4	16.2	40.6	43.2			6752	12,154
	5		48.4	51.6			8004	14,403

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* Type of analysis is denoted as follows:

1 - sample as received at laboratory
2 - moisture free
3 - moisture and ash free
4 - moist mineral matter free
5 - dry mineral matter free

** Data from Cady (Ref. 53).

Reference: Illinois Bureau of Labor Statistics, now published as the Annual Coal, Oil and Gas Report of the Illinois Department of Mines and Minerals, Illinois Coal Reports, 1882, 1970.

TABLE 2.5-7

MODIFIED MERCALLI INTENSITY (DAMAGE) SCALE OF 1931 (ABRIDGED)

I. Not felt except by a very few under especially favorable circumstances. (I, Rossi-Forel Scale)

II. Felt only by a few persons at rest, especially on upper floors of buildings. Delicately suspended objects may swing. (I to II, Rossi-Forel Scale)

III. Felt quite noticeably indoors, especially on upper floors of buildings, but not recognized by many people as an earthquake. Standing motorcars may rock slightly. Vibration like passing of truck. Duration estimated. (III, Rossi-Forel Scale)

IV. During the day felt indoors by many, outdoors by few. At night some awakened. Dishes, windows, doors disturbed; walls creaked. Sensation like heavy truck striking building. Standing motor cars rocked noticeably. (IV to V, Rossi-Forel Scale)

V. Felt by nearly everyone, many awakened. Some dishes, windows, etc., broken; a few instances of cracked plaster; unstable objects overturned. Disturbances of trees, poles, and other tall objects sometimes noticed. Pendulum clocks may stop. (V to VI, Rossi-Forel Scale)

VI. Felt by all, many frightened and may run outdoors. Some heavy furniture moved; a few instances of fallen plaster or damaged chimneys. Damage slight. (VI to VII, Rossi-Forel Scale)

VII. Everybody may run outdoors. Damage negligible in buildings of good design and construction; slight to moderate in well-built ordinary structures; considerable in poorly built or badly designed structures; some chimneys broken. Noticed by persons driving motorcars. (VIII, Rossi-Forel Scale)

VIII. Damage slight in specially designed structures; considerable in ordinary substantial buildings with partial collapse; great in poorly built structures. Panel walls thrown out of frame structures. Fall of chimneys, factory stacks, columns, monuments, walls. Heavy furniture overturned. Sand and mud ejected in small amounts. Changes in well water. Persons driving motorcars disturbed. (VIII+ to IX-, Rossi-Forel Scale)

TABLE 2.5-7 (Cont'd)

IX. Damage considerable in specially designed structures; well-designed frame structures thrown out of plumb; great in substantial buildings, with partial collapse. Buildings shifted off foundations. Ground cracked conspicuously. Underground pipes broken. (IX+, Rossi-Forel Scale)

X. Some well-built wooden structures destroyed; most masonry and frame structures destroyed with foundations; ground badly cracked. Rails bent. Landslides considerable from river banks and steep slopes. Shifted sand and mud. Water splashed (slopped) over banks. (X, Rossi-Forel Scale)

XI. Few, if any, (masonry) structures may remain standing. Bridges destroyed. Broad fissures in ground. Underground pipelines completely out of service. Earth slumps and landslips in soft ground. Rails bent greatly.

XII. Damage Total. Waves seen on ground surface. Lines of sight and level distorted. Objects thrown upward into the air.

TABLE 2.5-8

EARTHQUAKE EPICENTERS, 38° TO 46° NORTH LATITUDE 84° TO 94° WEST LONGITUDE

DATE	LOCATION	NORTH LATITUDE (°)	WEST LONGITUDE (°)	MAXIMUM INTENSITY (MM)	FELT AREA (mi^2)	REFERENCES*
1804 Aug. 24	Fort Dearborn, Ill.	42.0	87.8	VI-VII	30,000	1, 2, 3, 5
1818 Apr. 11	St. Louis, Mo.	38.6	90.2	III-IV	7,500	1
1819 Sept. 16	Randolph County, Ill.	38.1	89.8	IV	9,600	1
1819 Sept. 17	Randolph County, Ill.	38.1	89.8	III-IV		1
1827 July 5	St. Louis, Mo.	38.6	90.2	IV-V		1
1827 July 5	Grant County, Ky.	38.7	84.6	IV	15,000	1, 2
1827 July 5	New Albany, Ind.	38.3	85.8		165,000	1, 2
1827 July 6	Cincinnati, Ohio	39.1	84.5	IV		1
1827 Aug. 6	New Albany, Ind.	38.3	85.8	VI		1, 2, 3, 5
1827 Aug. 7	New Albany, Ind.	38.3	85.8	VI		1, 2, 3, 5
1827 Aug. 14	St. Louis, Mo.	38.6	90.2	III		1
1838 June 9	St. Louis, Mo.	38.5	90.3	VI	300	1

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*Key to references follows tabulated pages.

TABLE 2.5-8 (Cont'd)

DATE	LOCATION	NORTH LATITUDE (°)	WEST LONGITUDE (°)	MAXIMUM INTENSITY (MM)	FELT AREA (mi^2)	REFERENCES*
1843 Feb. 16	St. Louis, Mo.	38.6	90.2	IV-V	100,000	1
1883 Nov. 14	St. Louis, Mo.	38.6	90.2	IV	1,200	1
1883 Dec. 28	Bloomington, Ill.	40.5	87.0	III		16
1884 March 31	Preble County, Ohio	39.6	84.8	II		1
1884 Sept. 19	Allen County, Ohio	40.7	84.1	VI	125,000	1, 2, 8, 14
1884 Dec. 23	Anna, Ohio	40.4	84.2	III		1, 5, 14
1885 Dec. 26	Bloomington, Ill.	40.5	89.0	III		1
1886 March 1	Butlerville, Ind.	39.0	85.5	IV		1, 2
1886 Aug. 13	Indianapolis, Ind.	39.8	86.2	IV-V		1
1887 Feb. 6	Vincennes, Ind.	38.7	87.4	VI	75,000	1, 2, 3, 6, 7
1889 Sept.	Anna, Ohio	40.4	84.2	III		1, 8, 14
1891 July 26	Evansville, Ind.	38.0	87.6	VI		1, 2, 3, 6
1892	Anna, Ohio	40.4	84.2			1, 8, 14
1896 March 15	Sidney, Ohio	40.3	84.2	IV		1, 8, 14
1897 Oct. 31	Niles, Mich.	41.8	86.3			1

TABLE 2.5-8 (Cont'd)

DATE	LOCATION	NORTH LATITUDE (°)	WEST LONGITUDE (°)	MAXIMUM INTENSITY (MM)	FELT AREA (mi^2)	REFERENCES*
1903 Nov. 20	Morgantown, Ind.	39.4	86.3			1
1903 Dec. 11	Effingham, Ill.	39.1	88.5	II		1
1903 Dec. 31	Fairmont, Ill.	41.6	88.1			1
1905 March 13	Menominee, Mich.	45.0	87.7	V		1, 3
1905 April 13	Keokuk, Iowa	40.4	91.6	IV-V	5,000	1, 2, 3
1905 Aug. 22	Quincy, Ill.	39.9	91.4	II-III		1
1906 Feb. 23	Anabel, Mo.	39.7	92.4	III		1
1906 March 6	Hannibal, Mo.	39.7	91.4	IV		1
1906 April 22	Milwaukee, Wis.	43.0	87.9			1
1906 April 24	Milwaukee, Wis.	43.0	87.9			1
1906 May 8	Shelby County, Ind.	39.5	85.8	III-IV	600	1
1906 May 9	Columbus, Ind.	39.2	85.9	IV		1, 2, 3
1906 May 11	Petersburg, Ind.	38.5	87.3	V	1,200	1, 2, 3
1845	Putnam County, Ohio	41.1	84.2	II		1
1850 April 4	Louisville, Ky.	38.3	85.8	V		1, 2, 4
1854 Feb. 28	Lexington, Ky.	38.1	84.5	VI		15

TABLE 2.5-8 (Cont'd)

DATE	LOCATION	NORTH LATITUDE (°)	WEST LONGITUDE (°)	MAXIMUM INTENSITY (MM)	FELT AREA (mi^2)	REFERENCES*
1857 Oct. 8	St. Louis, Mo.	38.6	90.3	VI-VII	7,500	1, 3
1865	Le Sueur, Minn.	44.5	93.9	VI-VII		1, 2
1869 Feb. 20	Lexington, Ky.	38.1	84.5	III-IV		1
1871 July 25	St. Clair County, Ill.	38.5	90.0	III	1,000	1
1872 July 8	Chillicothe, Mo.	39.8	93.6	III		1
1873 April 22	Dayton, Ohio	39.8	84.2	III-IV		1
1875 June 18	Champaign County, Ohio	40.2	84.0	VII	40,000	1, 2, 6, 8, 14
1876 Jan. 27	Adrian, Mich.	41.9	84.0			1
1876 June	Anna, Ohio	40.4	84.2			1, 8, 14
1876 Sept. 24	Wabash County, Ill.	38.5	87.9	VI		1
1876 Sept. 25	Knox County, Ind.	38.5	87.7	VI	60,000	1, 2, 3, 6, 7
1876 Sept. 26	Wabash County, Ill.	38.5	87.9	III		1
1877 May 26	New Harmony, Ind.	38.1	87.9	III-IV		1
1881 April 20	Goshen, Ind.	41.6	85.8	IV		1
1881 May 27	La Salle, Ill.	41.3	89.1	VI		1, 2

TABLE 2.5-8 (Cont'd)

DATE	LOCATION	NORTH LATITUDE (°)	WEST LONGITUDE (°)	MAXIMUM INTENSITY (MM)	FELT AREA (mi^2)	REFERENCES*
1881 Aug. 29	Hillsboro, Ohio	39.2	83.6	III		1
1882 Feb. 9	Anna, Ohio	40.4	84.2	V	100	1, 2, 3, 8, 14
1882 July 20	Randolph County, Ill.	38.0	90.0	V	30,000	1, 2
1882 Sept. 27	Macoupin County, Ill.	39.0	90.0	VI	25,000	1, 2, 3
1882 Oct. 14	Macoupin County, Ill.	39.0	90.0	V	8,000	1, 2
1882 Oct. 15	Macoupin County, Ill.	39.0	90.0	V	8,000	1, 2, 3
1882 Oct. 22	Greenville, Ill.	38.9	89.4	III		1
1882 Nov. 15	St. Louis, Mo.	38.6	90.2	III		1
1883 Feb. 4	Kalamazoo County, Mich.	42.3	85.6	VI	150,000	1, 2, 3
1899 Feb. 8	Chicago, Ill.	41.9	87.6			1
1899 Feb. 9	Chicago, Ill.	41.9	87.6			1
1899 April 29	Dubois County, Ind.	38.5	87.0	VII	40,000	1, 2, 6, 7, 9
1899 Oct. 10	St. Joseph, Mich.	42.1	86.5	IV		1
1899 Oct. 12	Kenosha, Wis.	42.6	87.8			1
1902 Jan. 24	Maplewood, Mo.	38.6	90.3	VI	40,000	1, 3
1902 March 10	Hagerstown, Ind.	39.9	85.2	III-IV		1

TABLE 2.5-8 (Cont'd)

DATE	LOCATION	NORTH LATITUDE (°)	WEST LONGITUDE (°)	MAXIMUM INTENSITY (MM)	FELT AREA (mi^2)	REFERENCES*
1903 Jan. 1	Hagerstown, Ind.	39.9	85.2	II-III		1
1903 Feb. 8	St. Louis, Mo.	38.6	90.3	VI	40,000	1, 3
1903 March 17	Hillsboro, Ill.	39.2	89.5	III-IV		1
1903 Sept. 20	Morgantown, Ind.	39.4	86.3	IV		1
1903 Sept. 21	Olney, Ill.	38.7	88.1	IV		1
1903 Nov. 4	St. Louis, Mo.	38.6	90.3	VI-VII	70,000	1, 3
1909 May 26	South Beloit, Ill.	42.5	89.0	VII	170,000	1, 2, 3, 5
1909 July 18	Mason County, Ill.	40.2	90.0	VII	35,000	1, 2, 3
1909 Aug. 16	Monroe County, Ill.	38.3	90.2	IV-V	18,000	1
1909 Sept. 22	Lawrence County, Ind.	38.7	86.5	V	4,000	1, 2, 3
1909 Sept. 27	Robinson, Ill.	39.0	87.7	VII	30,000	1, 2, 3, 6, 10
1909 Sept. 27	Vincennes, Ind.	38.7	87.5	V	4,000	1, 2, 3, 6, 10
1909 Oct. 22	Sterling, Ill.	41.8	89.7	IV-V		1, 2
1909 Oct. 22	Near Scott, Ky.	38.9	84.5			1
1909 Oct. 23	Robinson, Ill.	39.0	87.7	V	14,000	1, 2, 5

TABLE 2.5-8 (Cont'd)

DATE	LOCATION	NORTH LATITUDE (°)	WEST LONGITUDE (°)	MAXIMUM INTENSITY (MM)	FELT AREA (mi^2)	REFERENCES*
1911 Feb. 28	St. Louis County, Mo.	38.7	90.3	IV		1
1911 July 29	Chicago, Ill.	41.9	87.6	??		1, 2
1912 Jan. 2	Kendall County, Ill.	41.5	88.5	VI	40,000	1, 3
1912 Sept. 25	Rockford, Ill.	42.3	89.1	??		1, 2
1906 May 19	Grand Rapids, Mich.	43.0	85.7	??		1
1906 May 21	Flora, Ill.	38.7	88.5	V	580	1, 2, 3, 6
1906 Aug. 13	Greencastle, Ind.	39.6	86.9	IV		1
1906 Sept. 7	Owensville, Ind.	38.3	87.7	IV	500	1
1906 Nov. 23	Anabel, Mo.	39.7	92.4	III		1
1907 Jan. 10	Menominee, Mich.	45.1	87.6			1
1907 Jan. 29	Morgan County, Ind.	39.5	86.6	V		1, 2
1907 Jan. 30	Greenville, Ill.	38.9	89.4	V		1
1907 Nov. 20	Stephenson County, Ill.	42.3	89.8	IV	100	1, 2
1907 Nov. 28	Stephenson County, Ill.	42.3	89.8	IV	100	1, 2
1907 Dec. 10	St. Louis, Mo.	38.6	90.2	IV		1
1908 Nov. 12	Sedalia, Mo.	38.7	93.2	IV	700	1

TABLE 2.5-8 (Cont'd)

DATE	LOCATION	NORTH LATITUDE (°)	WEST LONGITUDE (°)	MAXIMUM INTENSITY (MM)	FELT AREA (mi^2)	REFERENCES*
1913 Oct. 16	Sterling, Ill.	41.8	89.7	III-IV	4,000	1, 2
1913 Nov. 11	Louisville, Ky.	38.3	85.8	IV		1
1914 Oct.7	Madison, Wis.	43.1	89.4	IV		1
1914	Anna, Ohio	40.4	84.2	II		1, 8, 14
1915 April 15	Olney, Ill.	38.7	88.1	II-III	3,000	1
1916 Jan. 7	Worthingon, Ind.	39.1	87.0	III	3,000	1
1916 May 31	Madison, Wis.	43.1	89.4	II		1
1916	Clarke County, Iowa	41.1	93.8	II-III		1
1917 April 9	Jefferson County, Mo.	38.1	90.6	VI	200,000	1, 3
1918 Feb. 22	Shiawassee County, Mich.	42.9	84.2	IV		1
1918 July 1	Hannibal, Mo.	39.7	91.4	IV		1
1919 May 25	Knox County, Ind.	38.5	87.5	V	18,000	1, 2, 3, 6
1920 April 30	Centralia, Ill.	38.5	89.1	IV	4,000	1
1920 May 1	St. Louis County, Mo.	38.5	90.5	V	10,000	1, 3
1921 March 14	Crawfordsville, Ind.	40.0	86.9	IV	25,000	1

TABLE 2.5-8 (Cont'd)

DATE	LOCATION	NORTH LATITUDE (°)	WEST LONGITUDE (°)	MAXIMUM INTENSITY (MM)	FELT AREA (mi^2)	REFERENCES*
1921 Sept. 8	Waterloo, Ill.	38.3	90.2	IV	4,000	1
1921 Oct. 9	Waterloo, Ill.	38.3	90.2	III	3,000	1
1922 April 10	Monmouth, Ill.	40.9	90.7	II		1
1922 July 7	Fond du Lac, Wis.	43.8	88.5	V		1, 2
1923 March 8	Greenville, Ill.	38.9	89.4	III-IV	4,000	1
1923 Nov. 9	Tallula, Ill.	40.0	89.9	V	600	1, 2, 3
1925 Jan. 26	Waterloo, Iowa	42.5	92.3	II	200	1
1925 March 3	Evanston, Ill.	42.0	87.7	II-III		1
1925 April 4	Cincinnati, Ohio	39.1	84.5			1, 8, 14
1925 April 26	Vanderburgh County, Ind.	38.0	87.5	VI	100,000	1, 2, 3
1925 July 13	Edwardsville, Ill.	38.8	90.0	V		1
1925 Oct.	Anna, Ohio	40.4	84.2	II		1, 8, 14
1926 Oct. 3	Princeton, Ind.	38.4	87.6	III		1
1928 Jan. 23	Near Mount Carroll, Ill.	42.0	90.0	IV	400	1, 2
1928 March 17	St. Louis, Mo.	38.6	90.2	I		1
1928 Oct. 27	Shelby County, Ohio	40.4	84.1	III	100	1, 8, 14

TABLE 2.5-8 (Cont'd)

DATE	LOCATION	NORTH LATITUDE (°)	WEST LONGITUDE (°)	MAXIMUM INTENSITY (MM)	FELT AREA (mi^2)	REFERENCES*
1929 Feb. 14	Near Princeton, Ind.	38.3	87.6	III-IV	1,000	1
1929 March 8	Shelby County, Ohio	40.4	84.2	V	5,000	1, 2, 3, 6, 8, 14
1930 May 28	Near Hannibal, Mo.	39.7	91.3	III		1
1930 June 26	Near Lima, Ohio	40.5	84.0	IV		1, 8, 14
1930 June 27	Near Lima, Ohio	40.5	84.0	IV		1, 8, 14
1930 Aug. 8	Near Hannibal, Mo.	39.6	91.4	III-IV		1
1930 Sept. 20	Anna, Ohio	40.4	84.2	VI		1, 2, 3, 8, 11, 14
1930 Sept. 29	Sidney, Ohio	40.3	84.2	III		1, 8, 14
1930 Sept. 30	Anna, Ohio	40.3	84.3	VII		1, 2, 3, 8, 9, 14
1930 Oct.	Anna, Ohio	40.4	84.2	III-IV		1, 8, 14
1930 Dec. 23	Near St. Louis, Mo.	38.6	90.5	III-IV	1,000	1
1931 Jan. 5	Elliston, Ind.	39.0	86.9	V	500	1, 2, 3, 12
1931 March 21	Sidney, Ohio	40.3	84.2	III		1, 8, 14
1931 March 31	Shelby County, Ohio	40.4	84.1	III		1
1931 June 10	Malinta, Ohio	41.3	84.0	V	1,500	1, 8, 14

TABLE 2.5-8 (Cont'd)

DATE	LOCATION	NORTH LATITUDE (°)	WEST LONGITUDE (°)	MAXIMUM INTENSITY (MM)	FELT AREA (mi^2)	REFERENCES*
1931 Sept. 20	Anna, Ohio	40.4	84.2	VII	45,400	1, 2, 3, 8, 11, 12
1931 Oct. 8	Anna, Ohio	40.4	84.2	III		1, 8, 14
1931 Oct. 18	Madison, Wis.	43.1	89.4	III		1
1931 Dec. 17	St. Louis, Mo.	38.6	90.2	II		1
1931 Dec. 31	Petersburg, Ind.	38.5	87.3			1
1933 Feb. 22	Sidney, Ohio	40.3	84.2	III-IV	2,000	1
1933 Nov. 16	Grover, Mo.	38.6	90.6	III-IV	1,500	1
1933 Dec. 6	Stoughton, Wis.	42.9	89.2	IV	5,000	1, 2, 3
1934 Nov. 12	Rock Island, Ill.	41.5	90.5	VI	5,000	1, 3
1935 Jan. 5	Moline, Ill.	41.5	90.6	IV	200	1, 2
1935 Jan. 30	Harrison County, Mo.	40.5	94.0	III		1
1935 Feb. 26	Burlington, Iowa	40.8	91.2	III		1
1935 Oct. 29	Pike County, Ill.	39.6	90.8			1
1936 Oct. 8	Butler County, Ohio	39.3	84.4	III	700	1, 8, 14
1936 Dec. 25	Cincinnati, Ohio	39.1	84.5	III		1

TABLE 2.5-8 (Cont'd)

DATE	LOCATION	NORTH LATITUDE (°)	WEST LONGITUDE (°)	MAXIMUM INTENSITY (MM)	FELT AREA (mi^2)	REFERENCES*
1937 March 2	Anna, Ohio	40.4	84.2	VII	70,000	1, 2, 6, 8, 9, 12, 14
1937 March 3	Anna, Ohio	40.4	84.2	V		1, 2, 8, 11, 14
1937 March 3	Anna, Ohio	40.4	84.2	III	200	1, 8, 14
1937 March 8	Anna, Ohio	40.4	84.2	VII-VIII	150,000	1, 2, 3, 6, 8, 12, 14
1937 April 23	Anna, Ohio	40.4	84.2	III	200	1, 8, 14
1937 April 27	Anna, Ohio	40.4	84.2	III	200	1, 8, 14
1937 May 2	Anna, Ohio	40.4	84.2	IV		1
1937 June 29	Peoria, Ill.	40.7	89.6	II		1
1937 Aug. 5	Near St. Louis, Mo.	38.5	90.2	II-III		1
1937 Aug. 5	Granite City, Ill.	38.7	90.2	II		1
1937 Oct. 16	Cincinnati, Ohio	39.1	84.5	II-III		1
1937 Nov. 17	Near Centralia, Ill.	38.6	89.1	V	8,000	1, 2, 3, 6, 12
1938 Feb. 12	Porter County, Ind.	41.6	87.0	V	6,500	1, 2
1938 Nov. 7	Dubuque, Iowa	42.5	90.7			1, 2
1939 March 18	Near Jackson Center, Ohio	40.4	84.0	II	500	1, 8, 14

TABLE 2.5-8 (Cont'd)

DATE	LOCATION	NORTH LATITUDE (°)	WEST LONGITUDE (°)	MAXIMUM INTENSITY (MM)	FELT AREA (mi^2)	REFERENCES*
1939 June 17	Anna, Ohio	40.4	84.2	IV	400	1, 8, 14
1939 July 9	Anna, Ohio	40.4	84.2	II		1, 8, 14
1939 July 18	Escanaba, Mich.	45.7	87.1			1
1939 Aug. 1	Escanaba, Mich.	45.7	87.1			1
1939 Nov. 7	Escanaba, Mich.	45.7	87.1	II-III		1
1939 Nov. 23	Monroe County, Ill.	38.2	90.1	V	150,000	1, 3
1939 Nov. 24	Davenport, Iowa	41.6	90.6	II-III		1, 2
1940 Jan. 8	Louisville, Ky.	38.3	85.8	II-III		1
1940 May 27	Louisville, Ky.	38.3	85.8	III		1, 2
1940 Nov. 23	Monroe County, Ill.	38.2	90.1	VI	150,000	1
1941 Oct. 4	St. Louis, Mo.	38.6	90.2	I		1
1941 Nov. 15	Waterloo, Ill.	38.3	90.2	III		1
1942 Jan.	Winfield, Mo.	39.0	90.7	III		1
1942 Jan. 14	St. Louis, Mo.	38.6	90.2		600	1
1942 Jan. 29	St. Louis, Mo.	38.6	90.2			1

TABLE 2.5-8 (Cont'd)

DATE	LOCATION	NORTH LATITUDE (°)	WEST LONGITUDE (°)	MAXIMUM INTENSITY (MM)	FELT AREA (mi^2)	REFERENCES*
1942 Jan. 30	St. Louis, Mo.	38.6	90.2			1
1942 March 1	Kewanee, Ill.	41.2	89.9	IV-V	3,700	1, 2
1942 Nov. 17	East St. Louis, Ill.	38.6	90.2	III-IV	200	1
1942 Dec. 27	Maplewood, Mo.	38.6	90.3	II		1
1943 Feb. 9	Marinette County, Wis.	45.5	88.2	II-III		1
1943 Feb. 15	Escanaba, Mich.	45.7	87.1			1
1943 April 13	Louisville, Ky.	38.3	85.8	IV		1
1943 April 18	Waterloo, Ill.	38.3	90.2	I		1
1943 May 20	West Alton, Mo.	38.9	90.2	I		1
1943 May 24	West Alton, Mo.	38.9	90.2	I		1
1943 June 8	Webster Groves, Mo.	38.6	90.4	III-IV		1
1943 June 15	House Springs, Mo.	38.4	90.6	I		1
1943 June 18	House Springs, Mo.	38.4	90.6	I		1
1943 Sept. 14	Near St. Louis, Mo.	38.7	90.3	I		1
1944 March 16	Elgin, Ill.	42.0	88.3	II		1
1944 Sept. 25	St. Louis, Mo.	38.6	90.2	IV	25,000	1

TABLE 2.5-8 (Cont'd)

DATE	LOCATION	NORTH LATITUDE (°)	WEST LONGITUDE (°)	MAXIMUM INTENSITY (MM)	FELT AREA (mi^2)	REFERENCES*
1944 Nov. 13	Anna, Ohio	40.4	84.2	III	18,000	1, 4, 18
1944 Nov. 16	Escanaba, Mich.	45.7	87.1	II		1
1944 Dec. 10	Escanaba, Mich.	45.7	87.1	IV		1
1945 March 27	St. Louis, Mo.	38.6	90.2	II-III		1
1945 May 18	Escanaba, Mich.	45.7	87.1	II		1
1945 May 21	Near St. Louis, Mo.	38.7	90.2	III-IV		1
1946 Feb. 24	Centralia, Ill.	38.5	89.1	V	1,500	1, 2, 10
1946 Nov. 7	Washington County, Mo.	38.0	90.7	II-III		1
1947 March 16	Kane County, Ill.	42.1	88.3	IV		1
1947 May 6	Milwaukee, Wis.	43.0	87.9	IV-V	3,000	1, 2
1947 June 29	Near St. Louis, Mo.	38.4	90.2	VI	15,000	1, 3
1947 Aug. 9	Branch County, Mich.	42.0	85.0	VI	70,000	1, 2, 3
1948 Jan. 5	Centralia, Ill.	38.5	89.1	V	300	1, 13
1948 Jan. 15	Madison County, Wis.	43.2	89.7	IV-V		1
1948 April 20	Iowa City, Iowa	41.7	91.5	III-IV		1

TABLE 2.5-8 (Cont'd)

DATE	LOCATION	NORTH LATITUDE (°)	WEST LONGITUDE (°)	MAXIMUM INTENSITY (MM)	FELT AREA (mi^2)	REFERENCES*
1949 June 8	Ste. Genevieve, Mo.	38.0	90.1	III	300	1
1949 Aug. 11	Clayton, Mo.	38.7	90.3	II		1
1949 Aug. 26	Defiance, Mo.	38.6	90.8	II-III		1
1950 April 20	Dayton, Ohio	39.8	84.2	III		1, 8, 14
1951 Sept. 19	Near Florissant, Mo.	38.9	90.2	III-IV	1,200	1
1952 Jan. 7	Champaign County, Ill.	40.3	88.3	II-III		1
1953 Sept. 11	Near Roxana, Ill.	38.6	90.1	VI	6,000	1, 3
1953 Dec. 30	Centralia, Ill.	38.5	89.1	IV	1,200	1
1954 Aug. 9	Petersburg, Ind.	39.5	87.3	V		1, 2
1955 April 9	Near Sparta, Ill.	38.1	89.8	VI	20,000	1, 3
1955 May 29	Ewing, Ill.	38.1	88.9	III-IV		1
1956 Jan. 27	Anna, Ohio	40.4	84.2	V	2,000	1, 2, 8, 14
1956 March 13	Fulton County, Ill.	40.5	90.2	IV	2,000	1
1956 July 18	Oostburg, Wis.	43.6	87.8	IV		1
1956 Oct. 13	Near Milwaukee, Wis.	42.8	87.9	IV		1
1957 Jan. 8	Waupun, Wis.	43.6	88.7	III-IV		1

TABLE 2.5-8 (Cont'd)

DATE	LOCATION	NORTH LATITUDE (°)	WEST LONGITUDE (°)	MAXIMUM INTENSITY (MM)	FELT AREA (mi^2)	REFERENCES*
1958 Nov. 7	Wabash County, Ill.	38.4	87.9	VI	33,300	1, 2, 3, 9
1959 Jan. 6	St. Louis County, Mo.	38.8	90.4	II-III		1
1967 Feb. 2	Lansing, Mich.	42.7	84.5	IV		1
1967 Aug. 5	Jefferson County, Mo.	38.3	90.6	II		1
1968 Nov. 9	Hamilton County, Ill.	38.0	88.5	VII	585,000	1, 2, 3, 9
1968 Dec. 11	Louisville, Ky.	38.3	85.8	V		1
1971 Feb. 12	Wabash County, Ill.	38.5	87.9	IV	1,300	1
1972 Sept. 15	Lee County, Ill.	41.6	89.4	V-VI	40,000	1, 5
1973 April 18	St. Clair County, Ill.	38.5	90.2	II-III		1
1974 March 27	St. Louis, Mo.	38.5	90.1			17
1974 April 3	Southern Illinois	38.6	88.1	VI		17
1974 April 5	Eastern Missouri	38.6	90.9			17
1974 June 5	Kentucky	38.6	84.8			17
1974 June 5	Southern Illinois	38.6	89.9	V		17

TABLE 2.5-8 (Cont'd)

DATE	LOCATION	NORTH LATITUDE (°)	WEST LONGITUDE (°)	MAXIMUM INTENSITY (MM)	FELT AREA (mi^2)	REFERENCES*
1974 Aug. 22	Southern Illinois	38.2	89.7	V		17
1976 April 8	Stinesville, Idaho	39.3	86.8	V		18

TABLE 2.5-8 (Cont'd)

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TABLE 2.5-9

EARTHQUAKES OCCURRING OVER 200 MILES FROM THE SITE FELT AT THE BRAIDWOOD SITE

DATE	MODIFIED MERCALLI INTENSITY	LOCALITY	EPICENTER LOCATION (°N. LAT.)	(°W. LONG.)	FELT AREA (mi^2)	DISTANCE FROM SITE (mi)
1811 December 16	XI	Northeastern Arkansas Gulf Coast Tectonic Province	35.5	90.5	2,000,000	420
1812 January 23	X-XI	New Madrid, Missouri Gulf Coast Tectonic Province	36.6	89.5	2,000,000	330
1812 February 7	XI-XII	New Madrid, Missouri Gulf Coast Tectonic Province	36.6	89.5	2,000,000	330
1886 August 31	X	Charleston, South Carolina Atlantic Coast Tectonic Province	32.9	80.0	2,000,000	730
1895 October 31	VIII	Charleston, Missouri Gulf Coast Tectonic Province	37.0	89.4	1,000,000	300
1935 November 1	VI	Timiskaming, Canada Laurentian Shield Sub-Province of Central Stable Interior	46.8	79.1	1,000,000	580
1968 November 9	VII	Southern Illinois Central Stable Interior	38.0	88.5	580,000	225

TABLE 2.5-10

DIRECT SHEAR TEST DATA

PLANT SITE BORINGS

BORING NO.	ELEVATION (ft)	SOIL TYPE	FIELD MOISTURE CONTENT (%)	FIELD DRY DENSITY (lb/ft^3)	NORMAL PRESSURE (lb/ft^2)	YIELD STRENGTH (lb/ft^2)	PEAK STRENGTH (lb/ft^2)
A-2	582.0	SP	19.8	106	1000	615	920
A-3	587.3	SP	19.6	106	1000	625	940
A-3	557.3	SP	13.4	118	3500	2140	3200
A-5	568.5	SP	13.3	121	3000	2100	3150
A-6	590.2	SW.SP	18.1	110	1000	550	820

TABLE 2.5-11

STRENGTH TESTS*

COHESIVE SOILS

					TRIAXIAL COMPRESSION (UU)		UNCONFINED COMPRESSION
BORING NO.	ELEVATION (ft)	SOIL TYPE	FIELD MOISTURE CONTENT (%)	FIELD DRY DENSITY (lb/ft^3)	CONFINING PRESSURE (lb/ft^2)	SHEAR STRENGTH (lb/ft^2)	SHEAR STRENGTH (lb/ft^2)
MP-4	569.1	ML	18.5	111.9			2145
MP-7	572.4	ML	13.6	124.6	2400	6000	
MP-8	572.2	ML	8.5	136	2150	13680	
MP-8	567.2	ML	7.7	138	2600	9040	
MP-8	562.2	ML	14.1	118.8	3000	6800	
MP-10	571.8	ML	17.3	113.8	2360	1780	
MP-10	566.8	ML	17.7	112.7	2800	960	
MP-10	561.8	ML	7.7	137.8	3230	7000	
MP-12	574.3	ML	20.1	110.9	1900	3310	
MP-12	569.3	ML	7.1	139.9	2325	6980	
MP-12	564.3	ML	8.6	134.9	2750	11640	

TABLE 2.5-11 (Cont'd)

BORING NO.	ELEVATION (ft)	SOIL TYPE	FIELD MOISTURE CONTENT (%)	FIELD DRY DENSITY (lb/ft^3)	TRIAXIAL COMPRESSION (UU) CONFINING PRESSURE (lb/ft^2)	TRIAXIAL COMPRESSION (UU) SHEAR STRENGTH (lb/ft^2)	UNCONFINED COMPRESSION SHEAR STRENGTH (lb/ft^2)
MP-13	564.5	ML	8.6	136.2	2750	6560	
MP-15	573.2	ML	11.0	130.2	8640	13340	
MP-15	563.2	ML	8.5	136.0	8640	16760	
MP-15	559.5	ML	5.7	139.1	8640	12200	
MP-19	572.5	ML	19.5	113.9	8640	8920	
MP-19	567.5	ML	9.6	135.2	8640	8780	
MP-20	562.1	ML	11.3	131.2	2800	5740	
MP-24	577.3	ML	11.2	130.9	1500	9260	
MP-24	571.8	ML	9.3	132.3	1900	4560	
MP-24	566.8	ML	12.4	125.6	2400	1965	
MP-25	576.9	ML	9.0	135.2	1500	5000	
MP-25	571.9	ML	8.9	136.2	1970	10920	
MP-28	571.7	ML	13.9	118.0	8640	1250	
MP-28	566.7	ML	8.4	134.9	2800	9000	

TABLE 2.5-11 (Cont'd)

BORING NO.	ELEVATION (ft)	SOIL TYPE	FIELD MOISTURE CONTENT (%)	FIELD DRY DENSITY (lb/ft^3)	TRIAXIAL COMPRESSION (UU) CONFINING PRESSURE (lb/ft^2)	TRIAXIAL COMPRESSION (UU) SHEAR STRENGTH (lb/ft^2)	UNCONFINED COMPRESSION SHEAR STRENGTH (lb/ft^2)
MP-29	575.5	ML	9.8	132.7			9400
MP-29	564.5	ML	12.2	124.0			2600
MP-30	570.7	ML	10.9	127			9360*
MP-30	573.7	ML	11.0	140.3			7680*
MP-30	575.2	ML	12.0	131.3	60	6240*	
MP-30	563.2	ML	10.3	131			5520*
MP-30	566.2	ML	10.1	131			6960*
MP-35	582.6	ML	17.8	106.9			520
MP-35	562.6	ML	9.2	134.4			7060
MP-35	572.6	ML	8.4	135.6			11860
MP-40	581.8	ML	11.4	125.8	1500	6320	
MP-40	576.8	ML	10.2	131.8	1940	1360	
MP-40	571.8	ML	9.3	134.2	2380	10400	

TABLE 2.5-11 (Cont'd)

					TRIAXIAL COMPRESSION (UU)		UNCONFINED COMPRESSION
BORING NO.	ELEVATION (ft)	SOIL TYPE	FIELD MOISTURE CONTENT (%)	FIELD DRY DENSITY (lb/ft^3)	CONFINING PRESSURE (lb/ft^2)	SHEAR STRENGTH (lb/ft^2)	SHEAR STRENGTH (lb/ft^2)
MP-40	566.8	ML	9.3	134.2	2800	7960	
MP-42	575.1	ML	12.3	127.1			2760
MP-42	573.1	ML	10.3	132.5			800
MP-42	565.1	ML	9.9	131.3			1560
MP-56	577.5	ML	11.9	130.5	8640	8900	
MP-56	572.5	ML	10.4	132.0	8640	15840	
MP-56	567.5	ML	12.3	127.1	8640	3580	
MP-56	562.5	ML	9.1	134.1	2800	6400	
MP-65	571.6	ML	16.7	115.3	8640	3600	
MP-65	561.6	ML	12.9	122.6	8640	8860	
LSH-1	579.2	ML	9.7	134.1			13600
LSH-1	574.2	ML	9.2	133.7	8640	12660	

*Strength tests were performed on samples of 4-inch diameter.

TABLE 2.5-12

TRIAXIAL COMPRESSION (CU) TESTS

COHESIONLESS SOILS

BORING NO.	ELEVATION (ft)	SOIL TYPE	FIELD MOISTURE CONTENT (%)	FIELD DRY DENSITY (lb/ft^3)	CONFINING PRESSURE (lb/ft^2)	SHEAR STRENGTH* (lb/ft^2)
A-17	583.5	SP	19.4	106.7	1495	σ' tan 44
A-18	595.9	SP	20.2	99.8	1080	σ' tan 36

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[*] σ' = Effective Normal Stress.

TABLE 2.5-13

DENSITY TEST DATA

PLANT SITE

LOCATION	DEPTH (ft)	IN SITU DRY DENSITY (lb/ft^3)	ASTM D2049 MINIMUM DRY DENSITY (lb/ft^3)	ASTM D2049 MAXIMUM DRY DENSITY (lb/ft^3)	ASTM D1557 RELATIVE DENSITY (%)	ASTM D1557 MAXIMUM DRY DENSITY (lb/ft^3)
TP 1	3	103	90	112	64	---
	8	117	107	130	48	---
TP 2	3	104	87	110	78	---
	8	98	96	119	10.5	---
MP-14, MP-31	0-10	99	84	109	67	108
	10-17	103	91	110	67	105
	0-17	100	91	112	48	107
Plant Borings (Blended)	0-22	---	97	121	--	---
Ditch Bank (Bulk Samples)	0-5	---	89	111	--	110
	0-5	---	86	111	--	108
	5-10	---	91	113	--	110
	5-10	---	86	110	--	110
	10-15	---	93	112	--	107
	10-15	---	92	116	--	---

TABLE 2.5-14

SUMMARY OF BORINGS

BORING	SURFACE ELEVATION (ft)	DEPTH TO ROCK (ft)	BEDROCK SURFACE ELEVATION (ft)	TOTAL DEPTH OF HOLE (ft)
A-1	593.9	41.0	552.9	276.0
A-2	593.0	36.0	557.0	276.0
A-3	598.3	41.0	557.3	308.0
A-4	592.6	32.0	560.6	164.0
A-5	598.5	49.0	549.5	149.0
A-6	601.2	44.5	556.7	184.0
A-7	600.6	47.5	553.1	184.0
A-8	601.7	46.0	555.7	168.0
A-9	598.5	36.0	562.5	157.5
A-10	597.8	41.0	556.8	158.0
A-11	602.0	41.0	561.0	163.5
P-3	599.2	50.0	549.2	305.0
P-6	595.6	50.0	545.6	232.0
P-10	596.8	61.0	535.8	178.0
H-1	600.9	48.0	552.9	158.0
H-2	598.9	45.0	553.9	162.5
H-3	599.2	42.5	556.7	157.5
H-4	595.1	47.0	548.1	157.5
H-5	599.2	37.5	561.7	37.5
L-1	564.1	45.0	519.1	255.0
L-2	587.7	72.5	515.2	312.0
L-3	589.0	57.0	532.0	286.0
L-4	581.6	35.0	546.6	345.0
A-12	592.5	48.5	544.0	176.5
A-13	589.3	41.0	548.3	168.0
A-14	598.8	43.0	555.8	194.0
A-15	597.1	45.0	552.1	205.0
A-16	591.7	46.0	545.7	188.0
A-17	599.0	38.5	560.5	186.5
A-18	601.4	39.0	562.4	194.0
MP-1	598.0	45.0	553.0	164.0
MP-2	600.2	44.7	555.5	102.0
MP-3	600.5	40.7	559.8	112.0
MP-4	599.6	40.0	559.6	102.5
MP-5	601.6	42.0	559.6	103.0
MP-6	601.7	34.7	567.0	110.0
MP-7	602.9	41.2	561.7	110.0
MP-8	600.2	40.0	560.2	102.5

Note: All elevations refer to USGS datum.

TABLE 2.5-14 (Cont'd)

BORING	SURFACE ELEVATION (ft)	DEPTH TO ROCK (ft)	BEDROCK SURFACE ELEVATION (ft)	TOTAL DEPTH OF HOLE (ft)
MP-9	601.7	44.0	557.7	119.5
MP-10	602.3	44.0	558.3	102.0
MP-11	600.1	44.0	556.1	106.0
MP-12	599.3	42.0	557.3	104.0
MP-13	599.5	41.5	558.0	104.0
MP-14	599.4	44.0	555.4	164.0
MP-15	598.2	43.0	555.2	163.0
MP-16	598.8	40.0	558.8	105.0
MP-17	599.6	40.0	559.6	172.0
MP-18	598.9	36.5	562.4	190.0
MP-19	598.0	39.0	559.0	182.0
MP-20	597.6	41.0	556.6	107.0
MP-21	597.9	40.0	557.9	106.0
MP-22	599.3	39.0	560.3	110.0
MP-23	598.5	40.0	558.5	167.0
MP-24	597.8	38.0	559.8	157.0
MP-25	597.4	32.0	565.4	165.0
MP-26	599.3	40.0	559.3	169.0
MP-27	603.0	40.0	563.0	185.0
MP-28	602.2	41.0	561.2	190.5
MP-29	601.0	39.5	560.5	190.0
MP-30	601.2	39.5	561.7	190.0
MP-31	597.8	38.5	559.3	115.0
MP-32	603.3	40.0	563.3	118.0
MP-33	601.1	39.5	561.6	165.0
MP-34	600.0	39.5	560.5	174.0
MP-35	603.1	42.0	561.1	185.0
MP-36	602.6	41.5	561.1	169.0
MP-37	600.2	40.0	560.2	109.0
MP-38	602.7	40.0	562.7	185.0
MP-39	600.8	36.0	564.8	187.0
MP-40	602.3	40.5	561.8	195.0
MP-41	602.3	36.4	565.9	110.0
MP-42	603.1	40.0	563.1	189.5
MP-43	599.5	39.0	560.5	120.0
MP-44	599.5	40.0	559.5	110.0
MP-45	601.7	42.5	559.2	185.0
MP-46	598.9	36.5	562.4	110.0
MP-47	601.2	39.0	562.0	115.0
MP-48	599.5	39.5	560.0	186.0
MP-49	598.3	39.0	559.3	110.0
MP-50	601.7	40.0	561.7	120.0
MP-51	602.2	40.0	562.2	120.0
MP-52	601.8	40.0	561.8	181.0

TABLE 2.5-14 (Cont'd)

BORING	SURFACE ELEVATION (ft)	DEPTH TO ROCK (ft)	BEDROCK SURFACE ELEVATION (ft)	TOTAL DEPTH OF HOLE (ft)
MP-53	600.7	41.5	559.2	52.0
MP-54	598.6	38.5	560.1	49.0
MP-55	603.1	40.0	563.1	52.0
MP-56	603.0	45.0	558.0	46.5
MP-57	598.9	47.0	551.9	48.0
MP-58	599.3	39.0	560.3	41.0
MP-59	597.4	43.0	554.0	43.0
MP-60	594.6	-----	-----	40.0
MP-61	599.7	-----	-----	36.5
MP-62	600.0	34.9	656.1	35.5
MP-63	603.6	40.0	563.6	110.5
MP-64	599.8	40.0	559.8	98.5
MP-65	597.1	38.0	559.1	98.3
MP-66	597.7	39.7	558.0	87.5
MP-67	599.3	39.5	559.8	39.8
MP-68	602.2	42.2	560.0	42.5
LSH-1	599.7	33.5	566.2	95.0
HS-1	597.2	46.0	551.2	56.0
HS-2	598.1	44.0	554.1	54.0
HS-3	599.5	52.0	547.5	57.2
HS-4	598.3	50.0	548.3	60.0
HS-5	596.5	46.0	550.5	56.0
HS-6	597.7	42.0	555.7	52.0
HS-7	599.7	44.0	555.7	54.0
HS-8	599.3	43.5	555.8	53.5
HS-9	596.2	46.0	550.2	56.0
HS-10	598.8	49.0	549.8	59.0
HS-11	599.5	44.0	555.5	54.0
HS-12	598.2	43.5	554.7	53.5
HS-13	599.8	-----	-----	31.5
HS-14	597.9	-----	-----	27.0
HS-15	593.7	-----	-----	18.5
HS-16	598.3	-----	-----	24.5
HS-17	598.4	-----	-----	26.5
HS-18	590.1	-----	-----	14.0

TABLE 2.5-15

SUMMARY OF SHALLOW PIEZOMETER INSTALLATIONS

BORING	GROUND SURFACE ELEVATION (ft)	DEPTH TO BOTTOM OF WELL-POINT SCREEN (ft)	GEOLOGIC HORIZON	WATER ELEVATION (ft)	DATE
A-4	592.6	15.5	Dolton Member	590.9	11/1/72
				590.5	4/14/73
A-5	598.5	20.5	Dolton Member	587.0	11/1/72
				588.7	4/14/73
A-7	600.6	25.5	Dolton Member	590.7	11/1/72
				593.2	4/14/73
A-8	601.7	19.0	Dolton Member	594.7	11/1/72
				598.3	4/14/73
A-9	598.5	22.0	Dolton Member	590.1	11/1/72
A-10	597.8	27.0	Dolton Member	589.6	11/1/72
A-11	602.0	26.5	Dolton Member	592.2	11/1/72
H-1	600.9	24.0	Dolton Member	582.0	11/1/72
H-2	598.9	22.0	Dolton Member	580.3	11/1/72
				579.8	4/14/73
H-3	599.2	24.0	Dolton Member	583.1	11/1/72
H-4	595.1	17.5	Dolton Member	584.6	11/1/72
				585.8	4/14/73
P-10	596.8	52.0	Residual Soil	560.6	10/19/72

TABLE 2.5-15 (Cont'd)

BORING	GROUND SURFACE ELEVATION (ft)	DEPTH TO BOTTOM OF WELL-POINT SCREEN (ft)	GEOLOGIC HORIZON	WATER ELEVATION (ft)	DATE
MP-8	600.2	16.0	Dolton Member	596.1	4/24/73
MP-30	601.2	16.0	Dolton Member	598.3	4/24/73
MP-43	599.5	16.0	Dolton Member	599.0	4/24/73
MP-46	598.9	16.0	Dolton Member	597.1	4/24/73

TABLE 2.5-16

SUMMARY OF DEEP PIEZOMETER INSTALLATIONS

BORING	GROUND SURFACE ELEVATION (ft)	DEPTH OF SCREENED ZONE (ft)	GEOLOGIC HORIZON	WATER ELEVATION (ft)	DATE
A-1	593.9	Below 45	Carbondale* Formation	579.9	10/19/72
A-2	593.0	Below 50	Carbondale* Formation	579.4	10/19/72
A-3	598.3	Below 41	Carbondale* Formation	593.1 595.9	10/19/72 4/14/73
A-6	601.2	Below 50	Carbondale* Formation	588.2	10/19/72
H-2	598.9	148-158	Ft. Atkinson Limestone	536.1	4/14/73
H-4	595.1	147-157	Ft. Atkinson Limestone	530.7	4/14/73
P-3	601.4	Below 50	Carbondale* Formation	526.7	10/19/72
P-6	599.0	Below 50	Carbondale* Formation	543.6	10/19/72
Mine Vent Shaft***	599.08**			578.3	10/19/72

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*Uppermost formation.
**Elevation of reference point.
***Located in the SW 1/4 of the NE 1/4 of the NW 1/4 of Section 20, T.32N., R.9E.

TABLE 2.5-17

SEISMIC REFRACTION SURVEY:

SUMMARY OF COMPUTED DEPTHS AND CORRESPONDING COMPRESSIONAL WAVE VELOCITIES

SHOT POINT	d_0	V^1	d_1	V^2	d_2	V^3	d_3	V^4	d_4	V^5
0+00	0	1000	8.5	6000	51	8500	151	16,000		
2+50	0	1000	8.5	6000	51	8500	151	16,000		
5+00	0	1000	10	6000	39.5	8500	154.5	16,000		
7+50	0	1000	10	6000	44	8500	159	16,000		
10+00	0	1000	12	6000	46	8500	86*	10,000	201	16,000
10+00	0	1000	12	6000	46	8500	159	16,000		
12+00	0	1000	11	6000	53	8400	152	16,000		
15+00	0	1000	11	6000	49	8400	167.5	16,000		
17+50	0	1000	11	6000	50	8400	163.5	16,000		
20+00	0	1000	10	6000	39.5	8500	168	16,000		

SEISMIC LINE 2

0+50	0	1000	10.5	6000	50.5	9000	130	17,000
3+50	0	1000	11	6000	42	9000	153	17,000
5+50	0	1000	10.5	6000	54.5	9000	144.5	17,000
8+50	0	1000	9	6000	42.5	9000	153.5	17,000
10+00	0	1000	12.5	6000	52.5	9000	160.5	17,000
11+50	0	1000	16	6000	52	8500	190	17,000
12+50	0	1000	10	6000	50	9000	155	17,000
16+00	0	1000	9	6000	49	9000	171	17,000
18+00	0	1000	10.5	6000	42.5	9000	161	17,000
20+00	0	1000	11.5	6000	61.5	9000	167.5	17,000

TABLE 2.5-17 (Cont'd)

SHOT POINT	d_0	V^1	d_1	V^2	d_2	V^3	d_3	V^4	d_4	V^5
					SEIMIC LINE 1A**					
0+00	0	1000	15	7000	51	8000	154	20,000		
2+50	0	1000	13.6	6300	56	7400	130	20,400		
5+00	0	1000	10.6	6400	53	8000	122	14,000		
7+50	0	1000	12.2	6200	56	8500	127	14,000		
10+00	0	1000	10.1	6000	51	8700	141	15,500		

Notes:

1. For seismic line plan location, see Figure 2.5-55.

2. d = Depth, feet.

   V = Compressional wave velocity, feet per second.

(The subscript on depth indicates the top of a given layer, and on velocity, the compressional wave velocity for that layer.)

*Hidden layer case.

**Data were corrected to a 600-foot elevation datum.

TABLE 2.5-18

SURFACE WAVE DATA

OBSERVED WAVE	WAVE TYPE	PREDOMINANT PARTICLE MOTION	PREDOMINANT FREQUENCY (Hz)	APPARENT WAVELENGTH (ft)	APPARENT VELOCITY (ft/sec)	OBSERVED LENGTH OF WAVE TRAIN (cycles)
1	Rayleigh	Longitudinal Transverse	9.5	120	1150	4
2	Unknown	Longitudinal Transverse	13.5	47	635	6
3	Love	Longitudinal	10.0	52	520	5

TABLE 2.5-19

AMBIENT GROUND MOTION MEASUREMENTS
September 28, 1972)

AMBIENT STATION	FREQUENCY* (Hz)	GROUND MOTION**, X 10^{-3}		TRANS.	VERT.	LONG.
1	5.5, 8.5, 9.0, 11.0, 12.5, 14.5	Displacement	(in.)	.0055	.00125	.00375
(Near Boring A-6)	10.0, 12.5, 16.5	Acceleration	(in./s/s)	.192	.0417	.0917
	6.5, 7.0, 8.5, 9.0, 10.0, 11.0, 12.5, 14.5, 16.5	Velocity	(in./s)	.305	.0805	.255
2	10.0, 12.5	Displacement	(in.)	.00125	-	.00175
	9.0, 16.5	Acceleration	(in./s/s)	.0417	.025	
(Near Borings A-3) (See Figure 2.5-55)	6.5, 10.0, 12.5, 20.0, 25.0	Velocity	(in./s)	.140	.025	.110
3	4.5	Displacement	(in.)	.00075	-	.00075
	8.5, 12.5, 25.0	Acceleration	(in./s/s)	.025	.0333	.0166
(Near Boring A-1)	4.5, 5.0, 5.5, 10.0, 11.0, 12.5, 14.5, 16.5	Velocity	(in./s)	.175	.095	.125

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*Predominant frequencies are underlined.

**Trans. = transverse.
Vert. = vertical.
Long. = longitudinal.

TABLE 2.5-20

WATER-PRESSURE TEST RESULTS: BOREHOLE H-1

INTERVAL TESTED (ft)	VERTICAL DEPTH (ft)	DISCHARGE OF WATER LOSS (gpm)	GAUGE PRESSURE		WATER COLUMN PRESSURE (ft)	FRICTON HEAD LOSS (ft)	TOTAL HEAD (ft)	PERMEABILITY K	
			(psi)	(ft)				(ft/yr)	(cm/sec)
48.0 to 58.0	48.0	4.30	30.0	69.3	89.76	3.57	86.19	244.47	2.36-04
48.0 to 58.0	48.0	7.20	50.0	115.4	135.94	10.01	125.92	280.19	2.71-04
48.0 to 58.0	48.0	4.70	30.0	69.3	89.76	4.27	85.49	269.39	2.60-04
55.0 to 68.0	55.0	5.30	40.0	92.3	112.85	6.36	106.49	198.84	1.92-04
58.0 to 68.0	58.0	5.00	40.0	92.3	112.85	5.66	107.19	228.58	2.21-04
58.0 to 68.0	58.0	10.40	60.0	138.5	159.02	24.49	134.53	378.82	3.66-04
58.0 to 68.0	58.0	4.50	40.0	92.3	112.85	4.59	108.26	203.68	1.97-04
68.0 to 78.0	68.0	3.60	50.0	115.4	135.94	3.37	132.57	133.07	1.29-04
68.0 to 78.0	68.0	5.10	50.0	115.4	135.94	6.76	129.18	193.46	1.87-04
68.0 to 78.0	68.0	12.00	70.0	161.6	182.11	37.40	144.71	406.36	3.93-04
68.0 to 78.0	68.0	7.10	50.0	115.4	135.94	13.09	122.84	283.23	2.68-04
78.0 to 88.0	78.0	0.09	60.0	138.5	159.02	0.00	159.02	2.77	2.68-06
78.0 to 88.0	78.0	9.60	80.0	184.7	205.20	27.01	178.19	264.00	2.55-04
78.0 to 88.0	78.0	7.10	60.0	138.5	159.02	14.77	144.25	241.19	2.33-04
88.0 to 98.0	88.0	0.00	60.0	138.5	159.02	0.00	159.02	0.00	0.00
88.0 to 98.0	88.0	0.00	90.0	207.8	228.28	0.00	228.28	0.00	0.00
88.0 to 98.0	88.0	0.00	60.0	138.5	159.02	0.00	159.02	0.00	0.00
98.0 to 108.0	98.0	0.00	60.0	138.5	159.02	0.00	159.02	0.00	0.00
98.0 to 108.0	98.0	0.00	90.0	207.8	228.28	0.00	228.28	0.00	0.00
98.0 to 108.0	98.0	0.00	60.0	138.5	159.02	0.00	159.02	0.00	0.00
108.0 to 118.0	108.0	0.00	60.0	138.5	159.02	0.00	159.02	0.00	0.00
108.0 to 118.0	108.0	0.00	90.0	207.8	228.28	0.00	228.28	0.00	0.00
108.0 to 118.0	108.0	0.00	60.0	138.5	159.02	0.00	159.02	0.00	0.00
118.0 to 128.0	118.0	0.30	60.0	138.5	159.02	0.04	158.98	9.25	8.94-06
118.0 to 128.0	118.0	0.00	90.0	207.8	228.28	0.00	228.28	0.00	0.00
118.0 to 128.0	118.0	0.00	60.0	138.5	159.02	0.00	159.02	0.00	0.00
138.0 to 148.0	138.0	0.00	60.0	138.5	159.02	0.00	159.02	0.00	0.00
138.0 to 148.0	138.0	0.00	90.0	207.8	228.28	0.00	228.28	0.00	0.00
138.0 to 148.0	138.0	0.00	60.0	138.5	159.02	0.00	159.02	0.00	0.00

TABLE 2.5-20 (Cont'd)

INTERVAL TESTED (ft)	VERTICAL DEPTH (ft)	DISCHARGE OF WATER LOSS (gpm)	GAUGE PRESSURE (psi)	GAUGE PRESSURE (ft)	WATER COLUMN PRESSURE (ft)	FRICTON HEAD LOSS (ft)	TOTAL HEAD (ft)	PERMEABILITY K (ft/yr)	PERMEABILITY K (cm/sec)
128.0 to 148.0	128.0	0.00	60.0	138.5	159.02	0.00	159.02	0.00	0.00
128.0 to 148.0	128.0	0.50	90.0	207.8	228.28	0.12	228.16	6.22	6.01-06
128.0 to 148.0	128.0	0.30	60.0	138.5	159.02	0.04	158.98	5.35	5.18-06
148.0 to 161.0	148.0	0.00	60.0	138.5	159.02	0.00	159.02	0.00	0.00
148.0 to 161.0	148.0	0.00	90.0	207.8	228.28	0.00	228.28	0.00	0.00
148.0 to 161.0	148.0	0.00	60.0	138.5	159.02	0.00	159.02	0.00	0.00

Notes:

1. Gauge height above ground = 2.0 ft
2. Angle of hole = 0.0°
3. Diameter of hole = 3.0 in.
4. Depth of water table = 18.5 ft
5. Number of tests = 35
6. Length between packers = 10.0 ft
7. Diameter of pipe = 0.7 in.

TABLE 2.5-21

WATER-PRESSURE TEST RESULTS: BOREHOLE H-2

INTERVAL TESTED (ft)	VERTICAL DEPTH (ft)	DISCHARGE OF WATER LOSS (gpm)	GAUGE PRESSURE (psi)	GAUGE PRESSURE (ft)	WATER COLUMN PRESSURE (ft)	FRICTON HEAD LOSS (ft)	TOTAL HEAD (ft)	PERMEABILITY K (ft/yr)	PERMEABILITY K (cm/sec)
48.0 to 58.0	48.0	0.00	30.0	69.3	89.76	0.00	89.76	0.00	0.00
48.0 to 58.0	48.0	0.00	50.0	115.4	135.94	0.00	135.94	0.00	0.00
48.0 to 58.0	48.0	0.00	30.0	69.3	89.76	0.00	89.76	0.00	0.00
58.0 to 68.0	58.0	0.06	40.0	92.3	112.85	0.00	112.85	2.61	2.52-06
58.0 to 68.0	58.0	0.20	60.0	138.5	159.02	0.01	159.01	6.16	5.96-06
58.0 to 68.0	58.0	0.09	40.0	92.3	112.85	0.00	112.85	3.91	3.78-06
68.0 to 78.0	68.0	0.16	50.0	115.4	135.94	0.01	135.93	5.77	5.58-06
68.0 to 78.0	68.0	7.40	70.0	161.6	182.11	14.22	167.89	215.99	2.09-04
68.0 to 78.0	68.0	2.90	50.0	115.4	135.94	2.18	133.75	106.25	1.03-04
68.0 to 78.0	68.0	4.60	60.0	138.5	159.02	5.50	153.53	146.82	1.42-04
68.0 to 78.0	68.0	7.30	70.0	161.5	182.11	13.84	168.27	212.59	2.06-04
78.0 to 88.0	78.0	7.50	60.0	138.5	159.02	16.48	142.54	257.84	2.49-04
78.0 to 88.0	78.0	8.70	70.0	161.6	182.11	22.18	159.93	266.57	2.58-04
78.0 to 88.0	78.0	10.00	80.0	184.7	205.20	29.30	175.89	278.60	2.69-04
78.0 to 88.0	78.0	8.80	70.0	161.6	182.11	22.69	159.42	270.50	2.62-04
78.0 to 88.0	78.0	7.80	60.0	138.5	159.02	17.83	141.19	270.71	2.62-04
88.0 to 98.0	88.0	0.00	60.0	138.5	159.02	0.00	159.02	0.00	0.00
88.0 to 98.0	88.0	0.00	90.0	207.8	228.28	0.00	228.28	0.00	0.00
88.0 to 98.0	88.0	0.00	60.0	138.5	159.02	0.00	159.02	0.00	0.00
98.0 to 108.0	98.0	0.30	60.0	138.5	159.02	0.03	158.99	9.25	8.94-06
98.0 to 108.0	98.0	0.40	90.0	207.8	228.28	0.06	228.23	8.59	8.31-06
98.0 to 108.0	98.0	0.00	60.0	138.5	159.02	0.00	159.02	0.00	0.00
108.0 to 118.0	108.0	0.00	60.0	138.5	159.02	0.00	159.02	0.00	0.00
108.0 to 118.0	108.0	0.00	90.0	207.5	228.28	0.00	228.28	0.00	0.00
108.0 to 118.0	108.0	0.00	60.0	138.5	159.02	0.00	159.02	0.00	0.00
118.0 to 128.0	118.0	0.00	60.0	138.5	159.02	0.00	159.02	0.00	0.00
118.0 to 128.0	118.0	0.00	90.0	207.8	228.28	0.00	228.28	0.00	0.00
118.0 to 128.0	118.0	0.00	60.0	138.5	159.02	0.00	159.02	0.00	0.00
128.0 to 138.0	128.0	0.00	60.0	138.5	159.02	0.00	159.02	0.00	0.00

TABLE 2.5-21 (Cont'd)

INTERVAL TESTED (ft)	VERTICAL DEPTH (ft)	DISCHARGE OF WATER LOSS (gpm)	GAUGE PRESSURE (psi)	GAUGE PRESSURE (ft)	WATER COLUMN PRESSURE (ft)	FRICTON HEAD LOSS (ft)	TOTAL HEAD (ft)	PERMEABILITY K (ft/yr)	PERMEABILITY K (cm/sec)
128.0 to 138.0	128.0	0.00	90.0	207.8	228.28	0.00	228.28	0.00	0.00
128.0 to 138.0	128.0	0.00	60.0	138.5	159.02	0.00	159.02	0.00	0.00
138.0 to 148.0	138.0	0.00	60.0	138.5	159.02	0.00	159.02	0.00	0.00
138.0 to 148.0	138.0	0.00	90.0	207.8	228.28	0.00	228.28	0.00	0.00
138.0 to 148.0	138.0	0.00	60.0	138.5	159.02	0.00	159.02	0.00	0.00
148.0 to 158.0	148.0	0.00	60.0	138.5	159.02	0.00	159.02	0.00	0.00
148.0 to 158.0	148.0	0.00	90.0	207.8	228.28	0.00	228.28	0.00	0.00
148.0 to 158.0	148.0	0.00	60.0	138.5	159.02	0.00	159.02	0.00	0.00

Notes:

1. Gauge height above ground = 2.0 ft
2. Angle of hole = 0.0°
3. Diameter of hole = 3.0 in.
4. Depth of water table = 18.5 ft
5. Number of tests = 37
6. Length between packers = 10.0 ft
7. Diameter of pipe = 0.7000 in.

TABLE 2.5-22

WATER-PRESSURE TEST RESULTS: BOREHOLE H-3

INTERVAL TESTED	VERTICAL DEPTH	DISCHARGE OF WATER LOSS	GAUGE PRESSURE		WATER COLUMN PRESSURE	FRICTON HEAD LOSS	TOTAL HEAD	PERMEABILITY K	
(ft)	(ft)	(gal/min)	(psi)	(ft)	(ft)	(ft)	(ft)	(ft/yr)	(cm/sec)
45.5 to 52.5	45.5	0.00	25.0	57.7	76.72	0.00	76.72	0.00	0.00
45.5 to 52.5	45.5	0.00	40.0	92.3	111.35	0.00	111.35	0.00	0.00
52.5 to 62.5	52.5	1.50	30.0	69.3	88.26	0.47	87.79	83.72	8.10-05
52.5 to 62.5	52.5	7.40	50.0	115.4	134.43	11.40	123.04	294.72	2.85-04
52.5 to 62.5	52.5	2.10	30.0	69.3	88.26	0.92	87.34	117.82	1.14-04
62.5 to 72.5	62.5	1.20	45.0	103.9	122.89	0.35	122.54	47.99	4.64-05
62.5 to 72.5	62.5	6.80	60.0	138.5	157.52	11.16	146.36	227.67	2.20-04
62.5 to 72.5	62.5	2.20	45.0	103.9	122.89	1.17	121.72	88.57	8.56-05
72.5 to 82.5	72.5	1.60	50.0	115.4	134.43	0.70	133.73	58.63	5.67-05
72.5 to 82.5	72.5	5.80	70.0	161.6	180.61	9.24	171.37	165.85	1.60-04
72.5 to 82.5	72.5	3.30	50.0	115.4	134.43	2.99	131.44	123.03	1.19-04
82.5 to 92.5	82.5	1.40	50.0	115.4	134.43	0.60	133.83	51.26	4.96-05
82.5 to 92.5	82.5	2.10	70.0	161.6	180.61	1.36	179.25	57.41	5.55-05
82.5 to 92.5	82.5	1.50	50.0	115.4	134.43	0.69	133.74	54.96	5.31-05
92.5 to 102.5	92.5	8.10	55.0	127.0	145.98	22.39	123.58	321.18	3.11-04
92.5 to 102.5	92.5	13.50	75.0	173.2	192.15	62.21	129.94	509.09	4.92-04
92.5 to 102.5	92.5	10.60	55.0	127.0	145.98	38.35	107.63	482.62	4.67-04
102.5 to 112.5	102.5	11.50	55.0	127.0	145.98	49.55	96.43	584.37	5.65-04
102.5 to 112.5	102.5	14.40	75.0	173.2	192.15	77.68	114.47	616.45	5.96-04
102.5 to 112.5	102.5	11.60	55.0	127.0	145.98	50.41	95.57	594.79	5.75-04
112.5 to 122.5	112.5	12.40	60.0	138.5	157.52	62.72	94.80	640.98	6.20-04
112.5 to 122.5	112.5	14.10	75.0	173.2	192.15	81.10	111.05	622.18	6.02-04
112.5 to 122.5	112.5	11.40	60.0	138.5	157.52	53.02	104.51	534.54	5.17-04
122.5 to 132.5	122.5	0.60	60.0	138.5	157.52	0.16	157.36	18.68	1.81-05
122.5 to 132.5	122.5	1.10	80.0	184.7	203.70	0.53	203.16	26.53	2.57-05
122.5 to 132.5	122.5	0.90	60.0	138.5	157.52	0.36	157.16	28.06	2.71-05
132.5 to 142.5	132.5	0.20	65.0	150.1	169.07	0.02	169.05	5.80	5.61-06
132.5 to 142.5	132.5	0.50	85.0	196.2	215.24	0.12	215.12	11.39	1.10-05
132.5 to 142.5	132.5	0.30	65.0	150.1	169.07	0.04	169.02	8.70	8.41-06

TABLE 2.5-22 (Cont'd)

INTERVAL TESTED (ft)	VERTICAL DEPTH (ft)	DISCHARGE OF WATER LOSS (gal/min)	GAUGE PRESSURE (psi)	GAUGE PRESSURE (ft)	WATER COLUMN PRESSURE (ft)	FRICTON HEAD LOSS (ft)	TOTAL HEAD (ft)	PERMEABILITY K (ft/yr)	PERMEABILITY K (cm/sec)
142.5 to 152.5	142.5	0.30	65.0	150.1	169.07	0.05	169.02	8.70	8.41-06
142.5 to 152.5	142.5	0.40	85.0	196.2	215.24	0.08	215.16	9.11	8.81-06
142.5 to 152.5	142.5	0.20	65.0	150.1	169.07	0.02	169.05	5.80	5.61-06
152.5 to 162.5	152.5	0.30	70.0	161.6	180.61	0.05	180.56	8.14	7.87-06
152.5 to 162.5	152.5	1.00	90.0	207.8	226.78	0.54	226.24	21.66	2.09-05
152.5 to 162.5	152.5	0.70	70.0	161.6	180.61	0.27	180.34	19.02	1.84-05

Notes:

1. Gauge height above ground = 2.0 ft
2. Angle of hole = 0.0°
3. Diameter of hole = 3.0 in.
4. Depth of water table = 17.0 ft
5. Number of tests = 35
6. Length between packers = 10.0 ft
7. Diameter of pipe = 0.7 in.

TABLE 2.5-23

WATER-PRESSURE TEST RESULTS: BOREHOLE H-4

INTERVAL TESTED	VERTICAL DEPTH	DISCHARGE OF WATER LOSS	GAUGE PRESSURE		WATER COLUMN PRESSURE	FRICTON HEAD LOSS	TOTAL HEAD	PERMEABILITY K	
(ft)	(ft)	(gal/min)	(psi)	(ft)	(ft)	(ft)	(ft)	(ft/yr)	(cm/sec)
50.0 to 57.0	50.0	0.00	20.0	46.2	65.17	0.00	65.17	0.00	0.00
50.0 to 57.0	50.0	0.00	40.0	92.3	111.35	0.00	111.35	0.00	0.00
50.0 to 57.0	50.0	0.00	20.0	46.2	65.17	0.00	65.17	0.00	0.00
57.0 to 67.0	57.0	0.00	30.0	69.3	88.26	0.00	88.26	0.00	0.00
57.0 to 67.0	57.0	0.00	50.0	115.4	134.43	0.00	134.43	0.00	0.00
57.0 to 67.0	57.0	0.00	30.0	69.3	88.26	0.00	88.26	0.00	0.00
67.0 to 77.0	67.0	0.30	40.0	92.3	111.35	0.02	111.32	13.21	1.28-05
67.0 to 77.0	67.0	0.60	60.0	138.5	157.52	0.09	157.43	18.68	1.81-05
67.0 to 77.0	67.0	0.30	40.0	93.3	111.35	0.02	111.32	13.21	1.28-05
77.0 to 87.0	77.0	0.00	45.0	103.9	122.89	0.00	122.89	0.00	0.00
77.0 to 87.0	77.0	0.20	65.0	150.1	169.07	0.01	169.05	5.80	5.61-06
77.0 to 87.0	77.0	0.05	45.0	103.9	122.89	0.00	122.89	1.99	1.93-06
87.0 to 97.0	87.0	3.10	50.0	115.4	134.43	3.10	131.33	115.67	1.12-04
87.0 to 97.0	87.0	7.50	70.0	161.6	180.61	18.17	162.44	226.25	2.19-04
87.0 to 97.0	87.0	4.70	50.0	115.4	134.43	7.14	127.30	180.92	1.75-04
97.0 to 107.0	97.0	0.00	50.0	115.4	134.43	0.00	134.43	0.00	0.00
97.0 to 107.0	97.0	0.70	70.0	161.6	180.61	0.17	180.43	19.01	1.84-05
97.0 to 107.0	97.0	0.05	50.0	115.4	134.43	0.00	134.43	1.82	1.76-06
107.0 to 117.0	107.0	1.60	60.0	138.5	157.52	1.00	156.52	50.09	4.84-05
107.0 to 117.0	107.0	1.80	80.0	184.7	203.70	1.26	202.43	43.57	4.21-05
107.0 to 117.0	107.0	0.80	60.0	138.5	157.52	0.25	157.27	24.93	2.41-05
116.0 to 127.0	116.0	0.90	60.0	138.5	157.52	0.34	157.18	26.06	2.52-05
116.0 to 127.0	116.0	2.10	80.0	184.7	203.70	1.87	201.83	47.36	4.58-05
116.0 to 127.0	116.0	0.40	60.0	138.5	157.52	0.07	157.45	11.56	1.12-05
127.0 to 137.0	127.0	0.10	65.0	150.1	169.07	0.00	169.06	2.90	2.80-06
127.0 to 137.0	127.0	0.20	85.0	196.2	215.24	0.02	215.22	4.55	4.40-06
127.0 to 137.0	127.0	0.15	65.0	150.1	169.07	0.01	169.06	4.35	4.20-06
137.0 to 147.0	137.0	0.10	70.0	161.6	180.61	0.00	180.60	2.71	2.62-06

TABLE 2.5-23 (Cont'd)

INTERVAL TESTED (ft)	VERTICAL DEPTH (ft)	DISCHARGE OF WATER LOSS (gal/min)	GAUGE PRESSURE (psi)	GAUGE PRESSURE (ft)	WATER COLUMN PRESSURE (ft)	FRICTON HEAD LOSS (ft)	TOTAL HEAD (ft)	PERMEABILITY K (ft/yr)	PERMEABILITY K (cm/sec)
137.0 to 147.0	137.0	0.30	90.0	207.8	226.78	0.04	226.74	6.48	6.27-06
137.0 to 147.0	137.0	0.15	70.0	161.6	180.61	0.01	180.60	4.07	3.94-06
147.0 to 157.0	147.0	0.10	80.0	184.7	203.70	0.01	203.69	2.41	2.33-06
147.0 to 157.0	147.0	0.20	100.0	230.9	249.87	0.02	249.85	3.92	3.79-06
147.0 to 157.0	147.0	0.10	80.0	184.7	203.70	0.01	203.69	2.41	2.33-06

Notes:

1. Gauge height above ground = 2.0 ft
2. Angle of hole = 0.0°
3. Diameter of hole = 3.0 in.
4. Depth of water table = 17.0 ft
5. Number of tests = 33
6. Length between packers = 10.0 ft
7. Diameter of pipe = 0.7 in.

TABLE 2.5-24

SUMMARY OF PERMEABILITY TESTS

BORING	SAMPLE NO.	DEPTH (ft)	UNIFIED SOILS CLASSIFICATION	PERMEABILITY K (cm/sec)
H-5	1	0.5	SP	2.212×10^{-3}
	2	3.0	SP	1.784×10^{-3}
	3	5.5	SP	5.973×10^{-3}
	4	8.0	SP	5.647×10^{-4}
	5	13.0	SP	7.015×10^{-4}
	6	15.5	SP	6.2433×10^{-4}
	7	18.0	SP	7.826×10^{-4}
	8	20.5	SP	4.483×10^{-4}
	9	23.0	SP	3.658×10^{-4}
H-1	8	20.5	SP	8.3×10^{-3}
	12	35.5	GM	8.91×10^{-4}
	13	38.0	GM	8.24×10^{-4}
	14	40.5	GM	8.37×10^{-4}
H-2	6	13.0	SP	5.61×10^{-3}
	8	20.5	SP	4.74×10^{-3}
A-1	5	20.0	SP	7.37×10^{-2}
A-3	2	5.5	SP	5.41×10^{-2}
A-5	1	2.5	SP-ML	8.28×10^{-4}
	3	9.5	SP	8.84×10^{-4}
A-6	4	16.0	SP	9.68×10^{-4}

Note: Permeability tests were performed according to ASTM-D2434.

TABLE 2.5-24 (Cont'd)

BORING	SAMPLE NO.	DEPTH (ft)	UNIFIED SOILS CLASSIFICATION	PERMEABILITY K (cm/sec)
P-3	3	10.5	SP	7.72×10^{-4}
	4	16.0	SP	7.42×10^{-4}
P-6	2	5.5	SP	7.50×10^{-4}
	3	10.0	SP	7.27×10^{-4}
H-4	13	25.5	ML-SL	2.60×10^{-6}
H-3	14	37.0	GM	8.17×10^{-4}
HS-2	1	4.1	SM	1.7×10^{-3}
	2	8.0	SP	1.0×10^{-3}
	3	14.0	SP	4.1×10^{-3}
	4	19.0	SP	10.0×10^{-3}
HS-3	1	4.0	SP	2.6×10^{-3}
	2	9.0	SP	4.0×10^{-3}
	3	14.0	SP	7.0×10^{-3}
	4	19.0	SP	8.0×10^{-3}
	5	24.0	SP	7.0×10^{-3}
	6	27.8	SP	2.4×10^{-3}
HS-6	2	10.0	SP	2.4×10^{-3}

TABLE 2.5-25

SUMMARY OF PIEZOMETER READINGS

BORING OR TEST PIT	GROUND SURFACE ELEVATION (ft)	ELEVATION OF PIEZOMETER TIP (ft)	DATE MEASURED	WATER ELEVATION (ft)
H-1	600.9	576.9	10-29-72	581.9
	600.9	576.9	11-01-72	582.0
	600.9	576.9	11-22-72	581.0
	600.9	442.9	10-29-72	549.7
	600.9	442.9	11-22-72	537.2
H-2	598.9	576.9	10-29-72	582.7
	598.9	576.9	11-01-72	580.3
	598.9	576.9	11-22-72	579.6
	598.9	440.9	10-29-72	555.6
	598.9	440.9	11-22-72	533.4
H-3	599.2	575.2	10-29-72	583.0
	599.2	575.2	11-01-72	583.1
	599.2	575.2	11-22-72	582.5
	599.2	437.2	10-29-72	557.0
	599.2	437.2	11-22-72	556.5
H-4	595.1	577.6	10-29-72	584.6
	595.1	577.6	11-01-72	584.6
	595.1	577.6	11-22-72	584.5
	595.1	438.1	10-29-72	530.5
	595.1	438.1	11-22-72	529.2
HS-2	598.1			585.9
HS-3	599.5			587.9
HS-4	598.3			587.2
HS-5	596.5			587.0
HS-6	597.7			589.6
HS-7	599.7			587.5
HS-8	599.3			588.6
HS-11	599.5			591.0
HS-12	599.2			583.9
HS-13	599.8		02-18-75	585.8
HS-14	597.9		02-17-75	590.4
HS-15	593.7		02-18-75	584.0
HS-16	598.3		02-17-75	588.2
HS-17	598.4		02-17-75	589.7
HS-18	590.1		02-17-75	583.3

TABLE 2.5-25 (Cont'd)

BORING OR TEST PIT	GROUND SURFACE ELEVATION (ft)	ELEVATION OF PIEZOMETER TIP (ft)	DATE MEASURED	WATER ELEVATION (ft)
TP-1	590.1		02-27-75	583.9
TP-2	596.7		03-03-75	582.7
TP-4	594.1		02-28-75	585.0
TP-5	596.6		03-04-75	583.6
TP-6	595.4		03-13-75	582.5

TABLE 2.5-26

SUMMARY OF STATIC AND DYNAMIC PROPERTIES OF SUBSURFACE MATERIALS

	FORMATION					
PROPERTY	RECOMPACTED SANDS	WEDRON TILL	CARBONDALE SPOON BRAINARD	FORT ATKINSON	SCALES	WISE LAKE DUNLEITH
Approximate elevation (ft)	601 to 579	579 to 561	561 to 461	461 to 421	421 to 327	Below 327
Poisson's ratio (static or dynamic)	0.41	0.38	0.38	0.32	(0.32)*	0.32
Static modulus of elasticity, E (lb/ft^2)	0.2×10^6 to 1.0×10^6	0.9×10^6 to 5.0×10^6	0.1×10^8 to 0.5×10^8	3.5×10^8 to 7.5×10^8	1.5×10^8 to 3.5×10^8	8.0×10^8 to 10.0×10^8
Dynamic modulus of elasticity (lb/ft^2) single-amplitude shear strain – 1.0%	$5{,}600\ (\bar{\sigma}_m)^{0.5}$ **	0.3×10^6				
0.1%	$36{,}000\ (\bar{\sigma}_m)^{0.5}$	1.5×10^6				
0.01%	$127{,}000\ (\bar{\sigma}_m)^{0.5}$	5.5×10^6	0.8×10^8*** to 3.5×10^8	6.0×10^8*** to 11.0×10^8	2.0×10^8*** to 4.5×10^8	9.5×10^8 to 12.0×10^8***
Static modulus of rigidity, G (lb/ft^2)	0.07×10^6 to 0.4×10^6	0.4×10^6 to 2.0×10^6	0.1×10^8 to 0.2×10^8	1.5×10^8 to 3.0×10^8	0.6×10^8 to 1.5×10^8	3.0×10^8 to 4.0×10^8
Modulus number[τ]	500	5,000	50,000			
exponent[τ]	0.5	1.0	1.0	-	-	-

TABLE 2.5-26 (Cont'd)

	FORMATION					
PROPERTY	RECOMPACTED SANDS	WEDRON TILL	CARBONDALE SPOON BRAINARD	FORT ATKINSON	SCALES	WISE LAKE DUNLEITH
Dynamic modulus of rigidity (lb/ft^2)						
single-amplitude shear strain - 1.0%	2,000 $(\bar{\sigma}_m)^{0.5}$**	0.1×10^6				
0.1%	13,000 $(\bar{\sigma}_m)^{0.5}$**	0.5×10^6				
0.01%	45,000 $(\bar{\sigma}_m)^{0.5}$	2.0×10^6	0.3×10^8 to 1.0×10^8	2.0×10^8 to 4.0×10^8	0.7×10^8 to 2.0×10^8	3.5×10^8 to 4.5×10^8
Damping factor (percent of critical dumping)						
single-amplitude shear-strain - 1%	26%	20%				
0.1%	17%	15%				
0.01%	6%	10%	3%	2%	2%	2%

*Values in parenthesis are estimated values.

**$\bar{\sigma}_m$ is mean effective principal stress.
***These values represent the upper range of the deformation moduli and are valid for strain levels on the order of 10^{-4} to 10^{-5}%.
τNomenclature defined in Reference 69.

TABLE 2.5-27

DYNAMIC TRIAXIAL COMPRESSION TEST DATA

PLANT SITE BORINGS

COHESIONLESS SOILS

BORING NO.	ELEVATION (ft)	SOIL TYPE	MOISTURE CONTENT* (%)	DRY DENSITY* (lb/ft^3)	SINGLE AMPLITUDE SHEAR STRAIN (%)	MODULUS OF RIGIDITY (lb/ft^2)	DAMPING (%)
A-1	578.9	SP (Equality Formation)	22.3	105	0.0084	2.04x10^6	6
					0.0237	1.17x10^6	5
					0.0363	0.94x10^6	7
					0.0489	0.88x10^6	12
(Confining pressure = 1000 lb/ft^2)					0.0725	0.75x10^6	13
					0.1114	0.61x10^6	13
					0.1324	0.56x10^6	13
					0.293	0.31x10^6	24
					0.447	0.23x10^6	23
					0.571	0.19x10^6	25
					0.809	0.14x10^6	-
A-2	578.0	SP (Equality Formation)	22.3	105	0.0046	1.93x10^6	7
					0.0209	0.60x10^6	8
					0.0318	0.52x10^6	9
					0.0515	0.38x10^6	16
(Confining pressure = 1000 lb/ft^2)					0.0654	0.36x10^6	17
					0.2991	0.09x10^6	24
					0.591	0.08x10^6	24
					0.876	0.09x10^6	26

TABLE 2.5-27 (Cont'd)

BORING NO.	ELEVATION (ft)	SOIL TYPE	MOISTURE CONTENT* (%)	DRY DENSITY* (lb/ft^3)	SINGLE AMPLITUDE SHEAR STRAIN (%)	MODULUS OF RIGIDITY (lb/ft^2)	DAMPING (%)
A-3	583.3	SP	22.1	105	0.0117	$1.36x10^6$	7
		(Equality			0.0293	$0.76x10^6$	9
		Formation)			0.0400	$0.68x10^6$	17
					0.0714	$0.51x10^6$	18
(Confining pressure = 1000 lb/ft^2)					0.1026	$0.42x10^6$	21
					0.1275	$0.38x10^6$	20
					0.278	$0.22x10^6$	22
					0.581	$0.13x10^6$	24
					0.903	$0.10x10^6$	27
A-4	579.6	SP	20.8	107	0.0114	$1.91x10^6$	6
		(Equality			0.0226	$1.37x10^6$	7
		Formation)			0.0403	$0.97x10^6$	13
					0.0670	$0.82x10^6$	14
(Confining pressure = 1000 lb/ft^2)					0.0902	$0.71x10^6$	13
					0.1066	$0.71x10^6$	17
					0.259	$0.37x10^6$	21
					0.561	$0.20x10^6$	23
					0.847	$0.14x10^6$	27
MP-14	598.4	SM.SP	20.0	107	0.0072	$2.00x10^6$	15
and	to	(Equality			0.0281	$1.14x10^6$	16
MP-31	581.4	Formation)			0.0461	$0.88x10^6$	18
					0.1032	$0.41x10^6$	23
(Confining pressure = 2000 lb/ft^2)					0.1474	$0.26x10^6$	25
					0.2625	$0.11x10^6$	27
					0.5124	$0.03x10^6$	27

TABLE 2.5-27 (Cont'd)

BORING NO.	ELEVATION (ft)	SOIL TYPE	MOISTURE CONTENT* (%)	DRY DENSITY* (lb/ft^3)	SINGLE AMPLITUDE SHEAR STRAIN (%)	MODULUS OF RIGIDITY (lb/ft^2)	DAMPING (%)
MP-14	598.4	SM.SP	21.3	107	0.0118	2.57×10^6	7
and	to	(Equality			0.0276	2.07×10^6	8
MP-31	581.4	Formation)			0.0439	1.49×10^6	12
					0.0898	1.28×10^6	17
(Confining pressure = 6000 lb/ft^2)					0.1445	0.93×10^6	18
					0.3129	0.31×10^6	22
					0.6594	0.09×10^6	19

*Samples were recompacted in the laboratory to a relative density of at least 80%. The moisture content and dry density of the recompacted samples differ from in situ samples.

TABLE 2.5-28

DYNAMIC TRIAXIAL COMPRESSION TEST DATA

PLANT SITE BORINGS

COHESIVE SOILS

BORING NO.	ELEVATION (ft)	SOIL TYPE	FIELD MOISTURE CONTENT (%)	FIELD DRY DENSITY (lb/ft^3)	SINGLE AMPLITUDE SHEAR STRAIN (%)	MODULUS OF RIGIDITY (lb/ft^2)	DAMPING (%)
A-1	558.9	ML (Wedron Formation)	11.4	129	0.0064	2.46x10^6	17
					0.0014	1.63x10^6	14
					0.0312	0.92x10^6	11
					0.0598	0.60x10^6	22
(Confining pressure = 3,000 lb/ft^2, undrained shear strength = 8,280 lb/ft^2)					0.0734	0.55x10^6	22
					0.1029	0.44x10^6	25
					0.1717	0.32x10^6	24
					0.3041	0.20x10^6	26
					0.6426	0.11x10^6	23
					1.279	0.06x10^6	23
A-5	573.5	ML (Wedron Formation)	12.5	125	0.0128	0.98x10^6	15
					0.0342	0.86x10^6	14
					0.0517	0.67x10^6	11
					0.0876	0.47x10^6	16
(Confining pressure = 2,000 lb/ft^2, undrained shear strength = 7,620 lb/ft^2)					0.1408	0.35x10^6	24
					0.2083	0.27x10^6	-
					0.2978	0.20x10^6	26
					0.3841	0.16x10^6	-
					0.4669	0.13x10^6	21
					0.5033	0.11x10^6	21
					0.8207	0.08x10^6	25
					0.7945	0.06x10^6	-

TABLE 2.5-28 (Cont'd)

BORING NO.	ELEVATION (ft)	SOIL TYPE	FIELD MOISTURE CONTENT (%)	FIELD DRY DENSITY (lb/ft^3)	SINGLE AMPLITUDE SHEAR STRAIN (%)	MODULUS OF RIGIDITY (lb/ft^2)	DAMPING (%)
A-6	571.2	ML (Wedron Formation)	7.7	137	0.0043	16.27x10^6	-
					0.0057	13.86x10^6	-
					0.0074	14.94x10^6	14
					0.0194	6.68x10^6	11
(Confining pressure = 2,500 lb/ft^2, undrained shear strength = 12,770 lb/ft^2)					0.0479	4.50x10^6	15
					0.0758	3.55x10^6	23
					0.1019	3.11x10^6	29
					0.2293	1.77x10^6	32
					0.3805	1.53x10^6	32
					0.6773	0.97x10^6	-
					0.8135	0.89x10^6	-
MP-30	565.7	ML (Wedron Formation)	7.3	139	0.0148	5.90x10^6	7
					0.0342	3.86x10^6	9
					0.0767	2.55x10^6	8
(Confining pressure = 3,200 lb/ft^2, undrained shear strength = 25,490 lb/ft^2)					0.2060	2.29x10^6	12
					0.4193	1.75x10^6	10
MP-30*	563.2	ML (Wedron Formation)	9.7	134	0.0051	2.75x10^6	6
					0.0120	2.00x10^6	6
					0.0233	1.57x10^6	14
					0.0526	1.02x10^6	16
(Confining pressure = 4,100 lb/ft^2, undrained shear strength = 14,110 lb/ft^2)					0.0772	0.83x10^6	17
					0.1382	0.59x10^6	15
					0.2885	0.39x10^6	15
					0.5827	0.29x10^6	13

*Test conducted on 4-inch-diameter cored sample.

TABLE 2.5-29

FOUNDATION DATA

STRUCTURE	SEISMIC CATEGORY I	APPROXIMATE PLAN DIMENSIONS (ft)	APPROXIMATE FOUNDATION ELEVATION (ft)	APPROXIMATE STATIC BEARING PRESSURE (ksf)	BEARING STRATA	MAXIMUM ESTIMATED TOTAL SETTLEMENT* (in)
Reactor containment (Core)	Yes	160 (diam.)	562 (539)	6-10 ksf	Siltstone (sandstone)	0.5 inch (0.25 inch)
Auxiliary building	Yes	80 X 450 and 90 X 90	523 to 570	5-10 ksf	Siltstone	0.25 inch
Fuel handling building	Yes	90 X 120	579 and 595	4-5 ksf	Glacial till and recompacted sand	0.75 inch
Lake screen house	Yes	118 X 196	565	3 ksf	Glacial till	0.25 inch
Turbine building	No	130 X 740	551 to 596	4 ksf	Sandstone	0.5 inch
Turbine generator pedestal	No	60 X 210	562	6 ksf	Sandstone	0.5 inch
Radwaste building	No	90 X 140	594 and 598	2 ksf	Recompacted sand	0.5 inch
Service building	No	130 X 150	597	2 ksf	Recompacted sand	0.5 inch
Heater bay	No	50 X 500	591 and 597	4 ksf	Recompacted sand	1.0 inch

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[*] The maximum estimated settlements for the main power block structures were based on approximate static bearing pressure and approximate plan dimensions and elevations. The actual settlement values, based on field monitoring, are presented in Tables 2.5-41 and 2.5-42.

TABLE 2.5-30

IN-PLACE DENSITY AND WATER CONTENT TEST RESULTS FOR SAND DEPOSIT FROM UNDISTURBED TUBE AND BLOCK SAMPLES

SAMPLE DESIGNATION			UNIFIED	IN-PLACE DRY	WATER
BORING OR TEST PIT	SAMPLE NUMBER	DEPTH (ft)	SOIL CLASSIFCATION	DENSITY (lb/ft)	CONTENT (%)
HS-2	1	3	SP	99.4	
HS-2	1	4	SP	99.0	21.6
HS-2	1	4.5	SP	105.0	17.0
HS-2	2	8	SP	105.8	22.1
HS-2	2	9.5	SP	95.1	17.4
HS-2	3	14.0	SP	99.9	23.7
HS-2	3	14.5	SP	101.7	24.0
HS-2	4	18	SP	101.7	
HS-2	4	19	SP	101.6	22.2
HS-2	4	19.5	SP	105.4	21.8
HS-3	1	3	SP	99.8	
HS-3	1	4	SP	99.7	8.1
HS-3	1	4.5	SP	99.7	8.8
HS-3	2	8	SP	100.7	
HS-3	2	9	SP	93.2	21.2
HS-3	2	9.5	ML-SM	104.7	20.3
HS-3	3	14	SP	104.7	20.1
HS-3	3	14.5	SP	93.9	21.2
HS-3	4	19	SP	104.1	21.2
HS-3	4	19.5	SP	100.2	20.7
HS-3	5	23	SP	103.1	
HS-3	5	24	SP	105.8	20.5
HS-3	5	25	SP	97.6	20.3
HS-3	6	27.8	SM	113.1	13.1
HS-6	2	9	SP	107.5	
HS-6	2	10	SP	106.5	19.9
HS-6	2	10.5	SP	103.1	20.7
HS-16	U3	11.5	SM	103.9	22.6
HS-16	U3	12.3	SM	105.2	21.7
HS-18	U1	3.4	SM	104.1	21.9
HS-18	U1	3.9	SM	102.3	22.2
HS-18	U2	6.6	SP	97.8	25.2
TP-1*	1b	4.2	SM	108.3	11.4
TP-1*	2b	3.8	SM	106.0	11.7
TP-2*	11b	9.0	SM	98.7	6.9

TABLE 2.5-30 (Cont'd)

SAMPLE DESIGNATION BORING OR TEST PIT	 SAMPLE NUMBER	 DEPTH (ft)	UNIFIED SOIL CLASSIFCATION	IN-PLACE DRY DENSITY (lb/ft)	WATER CONTENT (%)
TP-3*	32b	5.6	SM	99.7	10.8
TP-3*	39b	11.2	SP	102.9	16.9
TP-4*	3b	8.1	SP	96.5	5.4
TP-5*	20b	11.2	SP-SM	101.8	15.9
TP-5*	21b	9.9	SM	103.4	14.2
TP-5*	49b	8.6	SM	100.6	8.4
TP-7*	52b	8.4	SP	103.2	4.7
TP-7*	59b	15.0	SP	98.8	4.3
TP-7*	60b	15.0	SP	95.3	5.5

*Indicates results obtained from block samples.

TABLE 2.5-31

RESULTS OF FIELD DENSITY TESTS IN SAND DEPOSIT BELOW ELEVATION 590

TEST PIT NO.	SAMPLE NO.	ELEV. (ft)	DEPTH (ft)	USC	FINES CONTENT (%)	D_{50} (mm)	WATER CONTENT (%)	γ_{max} (lb/ft^3)	γ_{min} (lb/ft^3)	γ_d (lb/ft^3)	D_r (%)
HTP-1	29	586.9	3.2	SM*	25	0.12	12.8	108.3	84.4	102.9	81
	2	586.3	3.8	SM*	15	0.16	11.7	114.8	94.5	109.5	77
	1	585.9	4.2	SM*	13	0.19	11.4	113.9	96.3	111.2	87
	30	584.5	5.6	SP	2	0.15	22.7	104.7	86.9	101.9	87
HTP-2	10	588.7	5.0	SM*	21	0.11	5.0	107.8	88.5	108.3	100
	11	587.7	9.0	SM*	21	0.10	6.9	105.0	86.1	103.6	94
	15	586.2	10.5	SP*	3	0.26	2.8	111.2	95.2	108.2	84
	14	585.3	11.4	SP	4	0.24	6.2	110.6	92.9	107.3	84
	13	584.7	12.0	SP	4	0.23	14.7	113.1	94.2	107.0	77
HTP-3	35	586.5	4.7	SM*	21	0.14	14.0	111.3	92.8	106.6	84
	32	585.6	5.6	SM*	13	0.16	10.8	104.1	89.4	104.0	99
	38	584.4	6.8	SP	1	0.27	3.7	106.1	91.7	105.9	99
	34	584.2	7.0	SP	1	0.25	3.6	106.5	91.8	103.2	80
	33	583.0	8.2	SP	1	0.26	3.8	106.5	91.8	103.7	83
	36	582.2	9.0	SP	2	0.22	6.2	106.5	91.7	106.2	98
	40	581.7	9.5	SP	1	0.28	5.0	107.2	93.6	106.8	97
	37	580.8	10.4	SP	1	0.24	5.6	105.6	91.5	105.4	99
	42	579.6	11.6	SP	1	0.26	16.9	110.2	93.8	107.1	83
	41	578.6	12.6	SP	3	0.18	21.3	108.7	89.2	104.8	83
HTP-4	1	589.7	4.3	SP-SM*	8	0.17	10.1	109.1	90.0	105.7	85
	4	589.0	5.0	SP*	2	0.25	3.7	109.3	91.4	103.0	69
	2	588.2	5.8	SP*	4	0.17	4.5	108.3	90.0	99.4	56
	5	586.8	7.2	SP	1	0.23	7.1	107.8	90.7	100.2	60
	3	585.9	8.1	SP	0	0.23	5.4	106.8	90.9	99.9	61

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*Denotes the test was made in brown silty fine sand.

TABLE 2.5-31 (Cont'd)

TEST PIT NO.	SAMPLE NO.	ELEV. (ft)	DEPTH (ft)	USC	FINES CONTENT (%)	D_{50} (mm)	WATER CONTENT (%)	Y_{max} (lb/ft^3)	Y_{min} (lb/ft^3)	Y_d (lb/ft^3)	D_r (%)
HTP-5	48	589.0	7.6	SP-SM[*]	12	0.17	6.3	110.5	91.7	100.5	51
	49	588.0	8.6	SM*	17	0.15	8.4	112.0	92.7	101.9	52
	23	587.3	9.3	SP-SM*	8	0.19	5.5	110.7	92.4	102.5	60
	21	586.7	9.9	SM*	21	0.14	14.2	111.5	92.3	106.6	78
	20	585.4	11.2	SP-SM	6	0.20	15.9	111.5	93.4	104.7	66
	22	584.4	12.2	SP-SM	11	0.16	19.2	113.0	92.9	108.8	79
HTP-6	46	587.5	7.9	SM*	44	0.09	8.3	110.7	93.8	109.5	94
	44	586.5	8.9	SP-SM*	10	0.22	3.2	112.1	95.1	106.2	69
	45	585.2	10.2	SP-SM*	11	0.19	9.0	111.6	94.5	113.4	100
	43	583.2	12.2	SP	2	0.24	16.2	106.1	91.4	106.5	100
HTP-7	52	588.4	8.4	SP*	2	0.22	4.7	105.6	89.0	104.7	95
	63	586.9	9.9	SM*	14	0.17	7.3	109.1	91.2	104.9	80
	55	585.6	11.2	SM*	35	0.09	19.4	108.1	85.4	102.0	78
	62	584.3	12.5	SP	4	0.25	3.8	107.4	94.0	103.5	74
	50	583.8	13.0	SP	1	0.23	7.2	105.3	91.8	104.4	94
	61	583.0	13.8	SP	2	0.21	4.2	104.9	91.3	101.3	76
	59	581.8	15.0	SP	3	0.22	4.3	108.0	91.8	104.1	79
	60	581.8	15.0	SP	2	0.25	5.5	108.1	91.6	100.6	76
	58	580.8	16.0	SP	1	0.21	5.5	104.4	91.0	104.7	100
	57	579.9	16.9	SP	2	0.19	12.6	105.5	91.6	101.2	85
	56	579.0	17.8	SP	1	0.22	21.3	105.1	85.4	103.6	94

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[*] Denotes the test was made in brown silty fine sand.

TABLE 2.5-32

RESULTS OF STATIC TESTS ON BROWN SILTY FINE SAND BELOW ELEVATION 590 FEET

PARAMETER	NO. OF DETERMINATIONS	AVERAGE
Water Content (%)	42	13.00
Grain Size:		
Finer than #40 sieve (%)	31	98.00
Finer than #60 sieve (%)	31	81.00
Finer than #100 sieve (%)	31	49.00
Finer than #200 sieve (%)	32	19.00
D_{50}, mm	31	0.16
Inferred relative density (%)	27	85.00
Maximum dry density (lb/ft^3)	21	109.80
Minimum dry density (lb/ft^3)	21	91.30
Field dry density (lb/ft^3)	21	105.00
Measured relative density (%)	21	80.00

TABLE 2.5-33

RESULTS OF FINE STATIC TESTS ON GRAY FINE SAND BELOW ELEVATION 585 FEET

PARAMETER	NO. OF DETERMINATIONS	AVERAGE
Water Content (%)	53	15.50
Grain Size:		
Finer than #40 sieve (%)	38	95.00
Finer than #60 sieve (%)	38	57.00
Finer than #100 sieve (%)	38	16.00
Finer than #200 sieve (%)	40	5.00
D_{50}, mm	38	0.24
Inferred relative density (%)	43	87.00
Maximum dry density (lb/ft^3)	20	107.20
Minimum dry density (lb/ft^3)	20	91.50
Field dry density (lb/ft^3)	20	104.80
Measured relative density (%)	20	87.00

TABLE 2.5-34

RESULTS OF STATIC TESTS ON GLACIAL TILL DEPOSITS

PARAMETER	NO. OF DETERMINATIONS	AVERAGE
Water Content (%)	64	13.4
Grain size finer than #200 sieve (%)	6	64.0
Liquid limit	8	23.7
Plastic limit	8	14.9

SAMPLE DESIGNATION						
BORING NO.	SAMPLE NO.	DEPTH (ft)	USC	IN-PLACE DRY DENSITY (lb/ft^3)	WATER CONTENT (%)	UNCONFINED COMPRESSIVE STRENGTH (lb/ft^2)
HS-3	6	28.5	CL-ML	131.0	11.5	7,200
HS-3	7	30.0	CL-ML	135.2	8.2	17,000
HS-6	4	20.6	CL	126.3	10.6	6,800

TABLE 2.5-35

RESULTS OF CYCLIC SHEAR STRENGTH

TESTS ON SAND DEPOSITS - PHASE 1

	TEST NO.	TYPE OF TEST SPECIMEN	D_{50} (mm)	FC (%)	γmin (lb/ft^3)	γmin (lb/ft^3)	γd (lb/ft^3)	D_r (%)	$\gamma 3_c$ (lb/ft^2)	γd	N (cycle) ± c				
										$2a_{3c}$	IL	2.5%	5%	7.5%	10%
TEST SERIES 1*	BR-101a	R	.25	1	104.8	91.6	101	74	1500	.426	13	16	19	24	44
	BR-101b	R	.25	1	104.8	91.6	101	74	1500	.397	18	17	25	30	40
	BR-101c	R	.25	1	104.8	91.6	101	74	1500	.497	6	8	10	12	18
	BR-101d	R	.25	1	104.8	91.6	101	74	1500	.443	11	15	19	24	37
	BR-102a	R	.25	1	104.8	91.6	103	88	1500	.457	15	20	28	62	68
	BR-102b	R	.25	1	104.8	91.6	103	88	1500	.491	11	16	23	39	47
	BR-102c	R	.25	1	104.8	91.6	103	88	1500	.404	30	37	44	64	81
	BR-103a	R	.25	1	104.8	91.6	98	52	1500	.398	4	5	7	8	10
	BR-103b	R	.25	1	104.8	91.6	98	52	1500	.378	12	14	16	17	19
	BR-103c	R	.25	1	104.8	91.6	98	52	1500	.361	12	13	15	16	18
	BR-103d	R	.25	1	104.8	91.6	98	52	1500	.388	7	8	10	11	14

TABLE 2.5-35 (Cont'd)

	TEST NO.	TYPE OF TEST SPECIMEN	D_{50}	FC	γmin	γmin	γd	D_r	$\gamma 3_c$	γd	N (cycle) ± c				
			(mm)	(%)	(lb/ft^3)	(lb/ft^3)	(lb/ft^3)	(%)	(lb/ft^2)	2a$_{3c}$	IL	2.5%	5%	7.5%	10%
TEST SERIES 2**	BR-104a	R	.15	11	110.9	92.1	110	93	1500	.494	30	36	51	102	108
	BR-104b	R	.15	11	110.9	92.1	110	93	1500	.470	30	39	58	105	110
	BR-104c	R	.15	11	110.9	92.1	110	93	1500	.425	58	74	115	174	195
	BR-105a	R	.15	11	110.9	92.1	107	80	1500	.494	17	21	26	38	66
	BR-105b	R	.15	11	110.9	92.1	107	80	1500	.451	21	26	33	48	80
	BR-105c	R	.15	11	110.9	92.1	107	80	1500	.426	31	36	43	70	88
	BR-106a	R	.15	11	110.9	92.1	103.5	63	1500	.488	10	12	14	16	25
	BR-106b	R	.15	11	110.9	92.1	103.5	63	1500	.447	11	13	15	19	33
TEST SERIES 3***	BR-107a	R	.17	20	112.8	92.9	109.2	85	1500	.494	26	31	46	95	106
	BR-107b	R	.17	20	112.8	92.9	109.2	85	1500	.447	35	41	50	108	128
	BR-107c	R	.17	20	112.8	92.9	109.2	85	1500	.425	35	35	54	125	172
	BR-108a	R	.17	20	112.8	92.9	106	70	1500	.489	14	17	22	29	68
	BR-108b	R	.17	20	112.8	92.9	106	70	1500	.426	25	29	35	47	77

TABLE 2.5-35 (Cont'd)

Key:

R: test on reconstituted test specimens.
D_{50}: 50% of sample is smaller than this grain size.
FC: fines content - percent passing the 1200 mesh sieve (0.074 mm).

* The test specimen for Test Series 1 was composed of material from Test Pit No. 3, block samples Nos. 37 and 38. Table 2.5-31 lists the elevations of the block samples.

** The test specimen for Test Series 2 was composed of material from Test Pit No. 3, block samples Nos. 32 and 37, and 38. Table 2.5-31 lists the elevations of the block samples.

*** The test specimen for Test Series 3 was composed of material from Test Pit No. 3, block samples Nos. 37 and 38 and Test Pit No. 5, block sample No. 21. Table 2.5-31 lists the elevations of the block samples.

TABLE 2.5-36

SUMMARY OF SKEMPTON "B" VALUES FOR PHASE 1 TESTS

TEST NUMBER	"B" VALUE
BR-101a	0.95
BR-101b	0.96
BR-101c	0.95
BR-102a	0.98
BR-102b	1.0
BR-102c	0.98
BR-103a	0.95
BR-103b	0.98
BR-103c	0.95
BR-103d	0.95
BR-104a	0.98
BR-104b	0.98
BR-104c	1.0
BR-105a	0.95
BR-105b	0.98
BR-105c	0.98
BR-106a	0.95
BR-106b	0.98
BR-107a	0.95
BR-107b	0.96
BR-107c	0.95
BR-108a	0.95
BR-108b	1.0

NOTE: The source of the test specimens referenced above is detailed in Table 2.5-35.

TABLE 2.5-37

RESULTS OF CYCLIC SHEAR STRENGTH TESTS ON SAND DEPOSITS - PHASE II

TEST NO.	TYPE OF TEST SPECIMEN	EL. (ft)	D_{50} (mm)	FC (%)	γ_{max} (lb/ft^3)	γ_{min} (lb/ft^3)	γ_d (lb/ft^3)	D_r (%)	$\sigma 3_c$ (lb/ft^2)	σd 2 $_{3c}$	N (cycle) ± c IL	 2.5%	 5%	 7.5%	 10%
BR-109a	I	585.9	0.16	16	113.9	96.3	108.3	72	1500	0.43	13	13	25	106	143
BR-109b	R	585.9	0.16	14	116.5	96.3	108.7	66	1500	0.42	10	12	14	17	39
BR-110a	I	586.3	0.18	17	114.8	94.5	105.2	58	1500	0.40	3	3	9	38	62
BR-110b	R	586.3	0.22	13	114.0	94.5	104.5	59	1500	0.41	16	16	18	21	31
BR-111a	I	586.3	0.17	17	114.8	94.5	106.8	65	1500	0.35	9	9	20	86	143
BR-111b	R	586.3	0.17	11	114.0	94.5	106.8	67	1500	0.36	20	22	26	30	51
BR-112a	I	586.3	0.17	17	114.8	94.5	106.7	65	750	0.40	4	6	13	48	81
BR-112b	R	586.3	0.17	14	114.0	94.5	106.8	67	750	0.42	30	30	34	38	60
BR-113a	I	585.9	0.20	1	106.8	90.9	96.5	39	1500	0.41	7	3	5	11	19
BR-113b	R	585.9	0.21	1	106.8	90.9	96.5	39	1500	0.41	7	7	9	12	16
BR-114a	I	585.4	0.15	13	111.5	93.4	101.8	51	1500	0.42	4	6	13	50	130
BR-114b	R	585.4	0.15	7	112.2	93.4	101.8	49	1500	0.40	4	5	6	7	9
BR-115a	I	587.7	0.09	20	105.0	86.1	98.7	71	1500	0.38	15	28	66	232	246
BR-115b	R	587.7	0.09	19	104.5	86.4	98.7	72	1500	0.38	25	28	31	34	39
BR-116a	I	586.7	0.12	33	111.5	92.3	103.4	62	1500	0.44	14	13	24	74	134
BR-116b	R	586.7	0.10	33	111.4	92.3	103.4	62	1500	0.43	6	7	8	10	11
BR-117a	I	580.2	0.22	3	108.2	93.1	102.9	68	1500	0.40	6	5	30	130	136
BR-117b	R	580.2	0.22	3	107.4	92.4	102	67	1500	0.40	9	11	14	15	36
BR-118a	I	588.0	0.16	14	112.0	92.7	100.6	46	1500	0.39	2	2	4	5	12
BR-118b	R	588.0	0.16	14	112.4	94.4	102	47	1500	0.39	6	6	7	8	9

TABLE 2.5-37 (Cont'd)

TEST NO.	TYPE OF TEST SPECIMEN	EL. (ft)	D_{50} (mm)	FC (%)	γ_{max} (lb/ft^3)	γ_{min} (lb/ft^3)	γ_d (lb/ft^3)	D_r (%)	$\sigma 3_c$ (lb/ft^2)	σd 2 $_{3c}$	N (cycle) ± c				
											IL	2.5%	5%	7.5%	10%
BR-119a	I	585.6	0.15	6	104.1	89.4	99.7	73	1500	0.41	12	22	114	165	169
BR-119b	R	585.6	0.17	7	105.4	89.4	99.7	68	1500	0.41	8	9	11	14	17
BR-120a	I	581.8	0.25	2	105.1	90.5	95.3	36	1500	0.35	3	3	5	10	39
BR-120b	R	581.8	0.25	3	105.1	90.5	95.3	36	1500	0.36	10	12	13	14	16
BR-121a	I	581.8	0.19	3	108.0	91.8	98.8	47	1500	0.45	4	4	8	27	68
BR-121b	R	581.8	0.18	4	109.0	91.8	98.8	45	1500	0.44	8	8	10	11	13
BR-122a	I	581.8	0.19	2	108.0	91.8	100	55	1500	0.49	3	3	7	44	65
BR-122b	R	581.8	0.16	3	109.0	91.8	100	52	1500	0.50	7	8	10	12	13

Key:

I: test on intact specimen.

R: test on reconstituted specimen.

D_{50}: 50% of the sample is smaller than this grain size.

FC: fines content = percent passing the #200 mesh sieve (0.074 mm).

TABLE 2.5-38

SUMMARY OF SKEMPTON "B" VALUES FOR PHASE II TESTS

TEST NUMBER	"B" VALUE
BR-109a	0.95
BR-109b	0.98
BR-109c	0.96
BR-110a	0.95
BR-110b	0.95
BR-111a	0.95
BR-111b	0.95
BR-112a	0.96
BR-112b	0.95
BR-113a	1.0
BR-113b	0.95
BR-114a	0.95
BR-114b	0.98
BR-115a	0.98
BR-115b	0.95
BR-116a	0.95
BR-116b	0.95
BR-116c	0.98
BR-117a	0.98
BR-117b	0.95
BR-117c	0.95
BR-118a	1.0
BR-118b	0.98
BR-118c	0.96
BR-119a	0.95
BR-119b	0.95
BR-119c	1.0
BR-120a	0.96
BR-120b	0.95
BR-121a	0.93
BR-121b	0.95
BR-122a	1.0
BR-122b	0.954

TABLE 2.5-39

SUMMARY OF RESULTS OF LIQUEFACTION

POTENTIAL ANALYSIS - C_r BASED ON D_r

CONDITION (D_r and FC)*	τ**	ELEV. (ft)	FACTOR OF SAFETY, τ_f/τ_d			
			BROWN SILTY FINE SAND		GRAY MEDIUM TO FINE SAND	
			MAXIMUM	MINIMUM	MAXIMUM	MINIMUM
Average	I.L.	588		1.17		
	± 5%			1.74		
	± 10%			2.36		
	I.L.	585	1.20			1.25
	± 5%		1.77			1.72
	± 10%		2.43			2.32
	I.L.	570			1.58	
	± 5%				2.18	
	± 10%				2.92	
Low average	I.L.	588		1.10		
	± 5%			1.51		
	± 10%			2.27		
	I.L.	585	1.13			1.05
	± 5%		1.55			1.43
	± 10%		2.31			1.85

TABLE 2.5-39 (Cont'd)

			FACTOR OF SAFETY, τ_f/τ_d			
CONDITION		ELEV.	BROWN SILTY FINE SAND		GRAY MEDIUM TO FINE SAND	
(D_r and FC)[*]	τ[**]	(ft)	MAXIMUM	MINIMUM	MAXIMUM	MINIMUM
	I.L.				1.34	
	± 5%	570			1.81	
	± 10%				2.32	

---

[*] D_r and FC for the brown silty fine and gray medium to fine sand are given with the corresponding strength curves on Figures 2.5-116 and 2.5-117.

[**] Strain criterion, I.L. = initial liquefaction.

TABLE 2.5-40

SUMMARY OF RESULTS OF LIQUEFACTION POTENTIAL ANALYSIS - C_r BASED ON K_o

			FACTOR OF SAFETY, τ_f/τ_d			
			BROWN SILTY FINE SAND		GRAY MEDIUM TO FINE SAND	
CONDITION (D_r and FC)*	τ**	ELEV. (ft)	MAXIMUM	MINIMUM	MAXIMUM	MINIMUM
Average	I.L.	588		1.30		
	± 5%			1.94		
	± 10%			2.63		
	I.L.	585	1.34		1.39	
	± 5%		1.97		1.91	
	± 10%		2.70		2.58	
	I.L.	570				1.40
	± 5%					1.92
	± 10%					2.57
Low average	I.L.	588		1.23		
	± 5%			1.68		
	± 10%			2.53		
	I.L.	585	1.26		1.17	
	± 5%		1.73		1.59	
	± 10%		2.57		2.06	

TABLE 2.5-40 (Cont'd)

CONDITION (D_r and FC)*	τ**	ELEV. (ft)	FACTOR OF SAFETY, τ_f/τ_d BROWN SILTY FINE SAND MAXIMUM	BROWN SILTY FINE SAND MINIMUM	GRAY MEDIUM TO FINE SAND MAXIMUM	GRAY MEDIUM TO FINE SAND MINIMUM
	I.L.					1.18
	± 5%	570				1.60
	± 10%					2.05

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[*] D_r and FC for the brown silty fine and gray medium to fine sand are given with the corresponding strength curves on Figures 2.5-116 and 2.5-117.

[**] Strain criterion, I.L. = initial liquefaction.

TABLE 2.5-41

TABULATED DIFFERENTIAL SETTLEMENTS FOR SURVEY MONUMENTS

BUILDING	MONUMENT NUMBER	PERIOD OF MEASUREMENT	MAXIMUM MEASURED DIFFERENTIAL MOVEMENT (feet)*	DIFFERENTIAL MOVEMENT BASED ON STABILIZED ELEVATION (ft)
Fuel	9	2/79 to 12/81	+0.002	-0.015
	10	2/79 to 8/80	-0.012	
	New 10	9/81 to 4/88	-0.006	
	New 9	9/81 to 12/85	-0.008	
	51	9/81 to 4/88	-0.004	
	52	9/81 to 6/83	+0.001	
	52 A	6/83 to 6/86	-0.011	
Refueling Water Storage Tanks	40	2/79 to 8/80	-0.025	-0.010
	New 40	9/81 to 4/88	-0.011	
	55	9/81 to 4/88	-0.006	
Auxiliary Building	KK	2/77 to 8/80	-0.059	-0.039
	LL	2/77 to 8/77	-0.013	
	JJ	2/77 to 5/77	-0.010	
	21	2/79 to 8/80	-0.020	-0.010
	22	2/79 to 8/80	-0.013	-0.010
	23	2/79 to 8/80	-0.015	-0.005
	24	2/79 to 8/80	-0.020	-0.015
	26	2/79 to 8/80	-0.021	-0.020
	27	2/79 to 8/80	-0.027	-0.020
	28	2/79 to 8/80	-0.025	
	New 21	9/81 to 1/87	-0.004	
	New 26	9/81 to 6/87	-0.004	
	New 27	9/81 to 3/87	-0.011	
	New 29	9/81 to 6/87	+0.002	
	53	9/81 to 3/87	+0.008	
	54	9/81 to 10/87	-0.002	

TABLE 2.5-41 (Cont'd)

BUILDING	MONUMENT NUMBER	PERIOD OF MEASUREMENT	MAXIMUM MEASURED DIFFERENTIAL MOVEMENT (feet)*	DIFFERENTIAL MOVEMENT BASED ON STABILIZED ELEVATION (ft)
Unit 1 Containment	U	2/77 to 8/80	-0.061	-0.070
	V	2/77 to 8/80	-0.052	-0.063
	N	2/77 to 8/80	-0.080	-0.067
	N2	3/77 to 6/77	-0.014	
	N4	3/77 to 6/77	-0.014	
	P	2/77 to 8/77	-0.004	
	13	2/79 to 2/80	-0.012	-0.008
	14	2/79 to 8/80	-0.005	-0.007
	15	2/79 to 8/80	-0.010	-0.012
	36	2/79 to 8/80	-0.003	-0.012
	39	2/79 to 8/80	-0.018	-0.012
	New U	9/81 to 3/86	-0.006	
	New V	9/18 to 10/82	+0.018 (Damaged)	
	New N	9/81 to 4/88	-0.010	
	New 3	9/81 to 3/87	-0.008	
	New 37	9/81 to 4/88	-0.008	
	New 39	9/81 to 4/88	-0.012	
Unit 1 Safety Valve Room	1 (Northeast Room)	2/79 to 8/80	-0.011	-0.015
	3 (Northwest Room)	2/79 to 8/80	-0.027	-0.025
Unit 2 Safety Valve Room	42	2/79 to 8/80	-0.024	-0.015

TABLE 2.5-41 (Cont'd)

BUILDING	MONUMENT NUMBER	PERIOD OF MEASUREMENT	MAXIMUM MEASURED DIFFERENTIAL MOVEMENT (feet)*	DIFFERENTIAL MOVEMENT BASED ON STABILIZED ELEVATION (ft)
Unit 2 Containment	AA	2/77 to 6/77	+0.005	
	BB	2/77 to 6/77	+0.006	
	R	2/77 to 8/77	-0.001	
	R1	2/77 to 8/77	-0.014	
	R2	2/77 to 5/77	-0.020	
	R3	2/77 to 8/77	-0.013	
	R4	2/77 to 8/80	-0.078	-0.074
	Z	2/77 to 8/80	+0.064	-0.065
	18	2/79 to 8/80	-0.020	-0.015
	19	2/79 to 8/80	-0.024	-0.018
	20	2/79 to 8/80	-0.020	-0.012
	43	2/79 to 8/80	-0.017	-0.008
	44	2/79 to 5/80	-0.007	-0.010
	Z1	9/81 to 10/86	-0.020 (Damaged)	
	New R4	9/81 to 6/87	-0.021	
	New 17	9/81 to 5/84	-0.001	
	New 18	9/81 to 4/88	-0.022	
	New 41	9/81 to 4/88	-0.023	
	New Z	9/81 to 6/87	-0.014	
Units 1 & 2 Turbine Room	CC	2/77 to 5/77	-0.001	
	HH	2/77 to 8/77	-0.033	
	T	2/77 to 8/77	-0.002	
	W	3/77 to 8/77	-0.013	
	X	2/77 to 8/77	+0.001	
	4	2/79 to 8/80	-0.010	-0.015

TABLE 2.5-41 (Cont'd)

BUILDING	MONUMENT NUMBER	PERIOD OF MEASUREMENT	MAXIMUM MEASURED DIFFERENTIAL MOVEMENT (feet)*	DIFFERENTIAL MOVEMENT BASED ON STABILIZED ELEVATION (ft)
	5	2/79 to 8/80	-0.001	-0.005
	6	2/79 to 8/82	+0.003	0
	33	2/79 to 8/82	-0.005	0
	New 4	9/81 to 1/88	+0.001	
	New 33	9/81 to 4/88	-0.012	
	New 34	9/81 to 4/88	+0.002	
	56	9/81 to 9/85	-0.012	
	58	9/81 to 4/88	+0.006	
	59	9/81 to 4/88	-0.011	
Heater Bay	57	9/81 to 6/87	-0.018	
Radwaste/Service Building	DD	2/77 to 8/77	-0.003	
	XX	2/77 to 8/80	-0.013	-0.023
	34	2/79 to 8/80	-0.008	0
Lake Screen House	60	1/84 to 1/88	+0.005	
	61	1/84 to 1/88	+0.007	
	62	1/84 to 4/88	+0	
	63	1/84 to 1/88	+0.006	
	64	1/84 to 4/88	+0.001	
	65	1/84 to 4/88	+0.003	

Key: - indicates downward movement for period of measurement given.
+ indicates upward movement for period of measurement given.

TABLE 2.5-42

PROJECTED MAXIMUM TOTAL AND DIFFERENTIAL SETTLEMENTS

CATEGORY I STRUCTURE	PROJECTED MAXIMUM[*] TOTAL SETTLEMENT (feet)	MAXIMUM DIFFERENTIAL SETTLEMENT (feet)
Unit 1 Containment	-0.074	-0.01
Unit 2 Containment	-0.078	-0.01
Auxiliary Building	-0.041	-0.03
Fuel Building	-0.04[**]	-0.02**
Refueling Water Tanks	-0.04**	-0.02**

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[*] Projected maximum total settlement determined by increasing by 5% the difference between stabilized monument elevations and the monument initial elevations. Monuments, U, V, Z, N, R_4, and KK were monitored from the beginning of construction to August 1980. These monuments were used to compute total settlement for the containments and auxiliary building areas.

[**] Settlement values given here are estimated conservatively because a significant amount of construction occurred before monuments were installed. Actual measurements indicate less than or equal to -0.025 feet total settlement.

TABLE 2.5-43

SUMMARY OF PERMEABILITY VALUES

BORING NUMBER	SAMPLE NUMBER	SAMPLE DEPTH (ft)	USCS	γ_d (pcf)	D_{10} (mm)	D_{20} (mm)	-#200 (%)	C_u	K (cm/sec)				REMARKS
									SEE NOTE 1	SEE NOTE 2	SEE NOTE 3	SEE NOTE 4	
HS-2	1	4.1	ML	99.0	0.019	0.035	83	2.84	$1.4\text{x}10^{-3}$	-	-	-	
	2	8.0	SM	105.8	-	-	27	-	$1.9\text{x}10^{-3}$	-	-	-	
	3	14.0	SM	99.9	-	0.074	19	-	$4.1\text{x}10^{-3}$	-	-	$0.9\text{x}10^{-3}$	
	4	19.0	SP	101.6	0.092	0.120	3	1.74	$10.0\text{x}10^{-3}$	$8.5\text{x}10^{-3}$	$9.1\text{x}10^{-3}$	$2.7\text{x}10^{-3}$	
HS-3	1	4.0	SM	9.87	-	0.075	19	-	$2.6\text{x}10^{-3}$	-	-	$0.9\text{x}10^{-3}$	
	2	9.0	SM	102.4	-	0.084	16	-	$4.1\text{x}10^{-3}$	-	-	$1.2\text{x}10^{-3}$	
	3	14.0	SP-SM	104.7	0.088	0.120	9	1.93	$7.0\text{x}10^{-3}$	$7.7\text{x}10^{-3}$	$8.1\text{x}10^{-3}$	$2.7\text{x}10^{-3}$	
	4	19.0	SP	104.1	0.149	0.170	3	1.68	$8.0\text{x}10^{-3}$	$2.2\text{x}10^{-2}$	$2.4\text{x}10^{-2}$	$6.1\text{x}10^{-3}$	
	5	24.0	SP	105.8	0.140	0.170	3	1.71	$7.0\text{x}10^{-3}$	$2.0\text{x}10^{-2}$	$2.1\text{x}10^{-2}$	$6.1\text{x}10^{-3}$	
	6	27.8	ML	113.1	0.004	0.011	72	-	$2.4\text{x}10^{-3}$	-		-	
HS-6	2	10.0	SM	106.5	-	$\simeq$0.070	22	-	$2.0\text{x}10^{-3}$			$\simeq 0.8\text{x}10^{-3}$	
H-1	8	20.5	SP						$8.3\text{x}10^{-3}$				
	12	35.5	GM						$0.9\text{x}10^{-3}$				See Note 5
	13	38.0	GM						$0.8\text{x}10^{-3}$				See Note 5
	14	40.5	GM						$0.8\text{x}10^{-3}$				See Note 5

TABLE 2.5-43 (Cont'd)

BORING NUMBER	SAMPLE NUMBER	SAMPLE DEPTH (ft)	USCS	γ_d (pcf)	D_{10} (mm)	D_{20} (mm)	-#200 (%)	C_u	K (cm/sec)				REMARKS
									SEE NOTE 1	SEE NOTE 2	SEE NOTE 3	SEE NOTE 4	
H-2	6	13.0	SP						5.6x10-3				
	8	20.5	SP						4.7x10-3				
H-3	14	37.0	GM						0.8x10-3				See Note 6
H-4	13	25.5	ML						2.6x10-6				

NOTES (N:):

1. Laboratory k values are from the tests performed on relatively undisturbed samples.
2. k values based on Hazen's Formula, k=100 D_{10}, cm/sec, where D_{10} in cm.
3. k values based on D_{10} (meters) and $C_u=D_{60}/D_{10}$. Reference: Beyer, W./Schweiger, "For the Determination of the Effective Porosity of Aquifers," Wasser Wirtsch, Wasser techn. 19, No. 2., 1969, pp. 57-60.
4. k values based on D_{20} (mm), k=0.36 D_{20}, cm/sec. Reference: Bialas, Z./Kleczkowski, A.S., "Practical Use of Certain Empirical Formulae to Determine Coefficient of Permeability k," Arch. Hydrotechn, Vol. 17, No. 3, 1970, pp. 405-417.
5. The sample was obtained from the Wedron Formation till. The GM layer is overlain by 5 ft. of sand and 3.5 ft of silty clay layer.
6. The sample was obtained from the Wedron Formation till. The GM layer is overlain by 11 feet of silt layer.

TABLE 2.5-44

BLAST DATA

BLAST	DATE AND TIME	BLAST LOCATION	TYPE OF BLAST	MAXIMUM BLAST LOADING (lb/delay)	MONITORING DISTANCE (ft)	BLAST MONITORING DATA	
						PEAK VELOCITY, (in/sec)	PEAK AIR PRESSURE, (lb/in^2)
A	12/31/75 4:45 p.m.	Unit 1	Presplit	40 to 128	≃1800	0.11	0.0006
B	12/31/75 4:50 p.m.	Unit 1	Presplit	106 to 110	≃1800	0.12	0.0011
C	01/06/76 4:41 p.m.	Unit 2	Presplit	40 to 96	≃1800	0.12	0.0028
D	01/06/76 4:48 p.m.	Unit 2	Presplit	40	≃1800	0.18	0.0019
E	01/07/76 4:26 p.m.	Unit 2	Production and Presplit	40 to 280	≃1800	0.50	0.0003
F	01/12/76 4:48 p.m.	Unit 1	Production and Presplit	120 to 260	≃2300	0.11	0.0026
G	01/12/76 4:36 p.m.	West of Unit 1 and 2	Production and Presplit	153	≃2800	0.04	Less than wind and background noise
H	01/22/76 4:30 p.m.	West of Unit 1 and 2	Production and Presplit	189	≃1900	0.15	-

NOTES

1. Presplit blasts utilized presplit explosives in the holes; individual holes were detonated with primacord surface line to down hole primacord lines; blasts detonated electrically.

2. Production blasts were loaded with conventional explosives, detonated by electric millisecond (ms) delay firing techniques. All explosive products used were manufactured by Atlas, except for Ensign-Bickford "Primacord."

TABLE 2.5-45

MATERIAL TESTING AND FREQUENCY

Field and laboratory test measurements shall be performed to the following minimum test frequencies.

TEST	FREQUENCY
FIELD DENSITY	
Controlled Compacted Fill	A, B, C, E, F, K, L, M
Regular Compacted Fill	A, B, (L*)
COMPACTION	
Controlled Compacted Fill	D, F, G, (L*), M
Regular Compacted Fill	F, J, D
MOISTURE CONTENT	
Borrow	C, D, H
Controlled Compacted Fill	C, H, K, (L*), M
Regular Compacted Fill	C, D, H
GRAIN SIZE	
Controlled Compacted Fill	F, J
Regular Compacted Fill	L
LIFT THICKNESS	
Controlled Compacted Fill	C, D, I
Regular Compacted Fill	C, D, I
RELATIVE DENSITY	
Controlled Compacted Fill	L

Key:

A = In areas where degree of compaction is doubtful.

B = In areas where earth fill operations are concentrated.

C = At least one for each earth fill shift.

D = One for every 8,000 yd^3 of fill for control and record.

E = For record tests at location of any embedded items.

TABLE 2.5-45 (Cont'd)

F = Where material identity is questionable.

G = One for each field density test as needed.

H = Where soil appears too wet or too dry.

I = Periodic surveillance and measurement checks.

J = One for every 4,000 yd^3 for record.

K = One for every 500 yd^3 for record and control (in confined areas only).

L = One for every 4,000 yd^3 for record and control.

M = One for every 500 linear feet of dike for slurry trench cap.

* Indicates a requirement for the Lake Work in addition to the listed requirements.

TABLE 2.5-46

ULTIMATE BEARING PRESSURES OF BACKFILL FOR CATEGORY I STRUCTURES AND BURIED PIPE

CATEGORY I STRUCTURE OR PIPELINE	AVERAGE ACTUAL ULTIMATE BEARING PRESSURE OF BACKFILL MATERIAL (ksf)	ULTIMATE BEARING PRESSURE OF THE FOUNDING STRATA (ksf)
Containment Building Unit 1	165	150
Containment Building Unit 2	170	150
Auxiliary Building	211	150
Fuel Handling Building	175.6	150
Essential Service Water Pipeline Foundation	75	45
Essential Service Water Pipeline Encasement	75.5	20

TABLE 2.5-47

MAXIMUM BEARING PRESSURE AND FACTORS OF SAFETY FOR ESSENTIAL SERVICE WATER DISCHARGE STRUCTURE

	STATIC LOADING		DYNAMIC LOADING			
			OBE		SSE	
	CASE I	CASE II	CASE I	CASE II	CASE I	CASE II
Maximum Bearing Pressure, ksf	1.22	1.26	1.42	1.46	1.66	1.70
Factors of Safety						
Against Sliding in Direction of Pipes (Section G, Figure 2.5-302)	5.3	5.0	6.4	6.4	5.9	5.9
Against Sliding in Direction Perpendicular to Pipe (Section F, Figure 2.5-302)	36.2	28.3	2.8	2.8	1.1	1.1
Against Overturning (Section G, Figure 2.5-302)	2.4	2.5	2.0	2.1	1.6	1.6
Against Overturning (Section F, Figure 2.5-302)	2.6	2.8	1.8	1.9	1.2	1.3

Note: Case I - Water Surface at Elevation 598.2 feet (Flood Conditions)
Case II - Water Surface at Elevation 587.0 feet

TABLE 2.5-48

ESSENTIAL SERVICE WATER PIPES - SUBGRADE AND BACKFILL CONDITIONS

PIPE LOCATION	SUBGRADE	BACKFILL
E line turbine wall to 5' west of C line turbine wall at 31+35 S, 43+85 E (4 pipes).	Pipeline subgrade consisted of fill concrete placed over Wedron silty clay till to bring grade level FSAR 2.5.4.1 and Figs. 2.5-16 & 2.5-25.	Reinforced concrete to 1 foot over pipe. Medium-fine sand above concrete compacted to 85% RD.
31+35 S, 43+85 E to 33+06.5 S (90 degree elbow) (4 pipes).	Pipeline subgrade consisted of Wedron silty clay till FSAR 2.5.4.5.1 Figs 2.5-16 & 2.5-25 Except in circ. water intake pipe excavation where fill concrete was placed between the circ. water and ESW pipes.	Pipes encased in lean concrete or Bash. Concrete encasement backfilled to grade to 80% RD min.
33+06.5 S to 33+92.5 S (4 pipes)	Pipeline subgrade consisted of medium-fine sand backfill compacted to 85% RD within circ. water discharge pipe excavation.	Encased in concrete backfilled with medium-fine sand and compacted to 85% RD min. to top of concrete and 80% RD min. above concrete.
33+92.5 S to 49+20 S (4 pipes).	Wedron Silty Clay Till FSAR 2.5.4.5-1 Figs. 2.5-16 & 2.5-25	Pipes encased in lean concrete or Bash. Concrete encasement backfilled to grade to 80% RD min.

TABLE 2.5-48 (Cont'd)

PIPE LOCATION	SUBGRADE	BACKFILL
49+20 S to 50+90 S (Screenhouse) (2 pipes).	Wedron Silty Clay Till FSAR 2.5.4.5.1 Figs 2.5-16 & 2.5-25	Pipes encased in lean concrete or Bash and backfilled to grade to 80% RD min.
49+20 S to 51.06 S (2 pipes).	Wedron Silty Clay Till FSAR 2.5.4.5.1 Figs 2.5-16 & 2.5-25	Pipes encased in lean concrete or Bash and backfilled to grade to 80% RD min.
51.06 S to 51.14 S (2 pipes).	Wedron Silty Clay Till FSAR 2.5.4.5.1 Figs 2.5-16 & 2.5-25	Lean concrete or Bash encasement (S-93BR) and backfilled with sand compacted to 80% RD min.
51.14 S to 52+00 S (2 pipes).	Wedron Silty Clay Till FSAR 2.5.4.5.4.1 Figs 2.5-16 & 2.5-25	Lean concrete or Bash encasement and backfilled with sand to 80% RD min.
52+00 S to 81+17.25 S	Pipeline encasement supported on pads founded on Wedron Silty Clay Till - 2.5.4.5.4.1.	Encased in lean concrete. Backfilled with sand to 85% min. RD on the sides of the pipes.

TABLE 2.5-49

FS AGAINST LIQUEFACTION FOR AVERAGE RELATIVE DENSITY CONDITIONS

(Level Ground at Elevation 590.0 ft)

(1) Elev	(2) Soil	(3) τ_d (psf)	(4) Strain Condition	(5) σ_v (psf)	(6) $\left(\frac{\sigma_d}{2\sigma_{3c}}\right)$ N=10	(7) τ_f (psf)	(8) FS	(9) D_c (based on D_r)	(10) τ_f (psf)	(11) FS (C_r based on D_r)	(12) C_r (based on K_o)	(13) τ_f (psf)	(14) FS (C_r based on K_o)
588	Brown	48.2	IL	134	0.56	56.3	1.17	1.00	56.3	1.17	0.83	62.3	1.30
	Fine	48.2	±5%	134	0.70	70.5	1.46	1.19	83.7	1.74	0.83	92.7	1.94
	Silty	48.2	±10%	134	0.83	83.5	1.72	1.38	114.0	2.36	0.83	127.0	2.63
	Sand												
585	Brown	115.0	IL	335	0.56	141.0	1.22	1.00	140.3	1.22	0.83	154.1	1.34
	Fine	115.0	±5%	335	0.70	176.0	1.53	1.19	203.6	1.77	0.83	226.6	1.97
	Silty	115.0	±10%	335	0.83	208.0	1.80	1.38	280.0	2.43	0.83	310.0	2.70
	Sand												
585	Gray	115.0	IL	335	0.54	135.8	1.18	1.03	143.8	1.25	0.83	159.9	1.39
	Fine	115.0	±5%	335	0.63	158.0	1.37	1.25	196.7	1.72	0.83	219.6	1.91
	Sand	115.0	±10%	335	0.75	188.5	1.64	1.42	266.0	2.32	0.83	296.0	2.58
570	Gray	375.9	IL	1340	0.54	543.0	1.48	1.03	581.4	1.58	0.65	515.2	1.40
	Fine	375.9	±5%	1340	0.63	633.0	1.72	1.25	802.2	2.18	0.65	706.6	1.92
	Sand	375.9	±10%	1340	0.75	755.0	2.05	1.42	1070.0	2.92	0.65	945.0	2.57

TABLE 2.5-50

FACTOR OF SAFETY (FS) AGAINST LIQUEFACTION FOR LOW AVERAGE RELATIVE DENSITY CONDITIONS

(Level Ground at Elevation 590.0 ft)

(1) Elev	(2) Soil	(3) τ_d (psf)	(4) Strain Condition	(5) σ_v (psf)	(6) $(\frac{\sigma_d}{2\sigma_{3c}})$ N=10	(7) τ_f (psf)	(8) FS	(9) D_c (based on D_r)	(10) τ_f (psf)	(11) FS (C_r based on D_r)	(12) C_r (based on K_o)	(13) τ_f (psf)	(14) FS (C_r based on K_o)
588	Brown	48.2	IL	134	0.53	53.3	1.11	0.99	53.0	1.10	0.83	59.3	1.23
	Fine	48.2	±5%	134	0.62	62.3	1.29	1.17	72.8	1.51	0.83	81.0	1.68
	Silty	48.2	±10%	134	0.79	79.5	1.65	1.36	109.4	2.27	0.83	121.9	2.53
	Sand												
585	Brown	115.0	IL	335	0.53	133.0	1.13	0.99	130.0	1.13	0.83	144.9	1.26
	Fine	115.0	±5%	335	0.62	156.0	1.36	1.17	178.2	1.55	0.83	199.0	1.73
	Silty	115.0	±10%	335	0.79	198.5	1.72	1.36	265.6	2.31	0.83	295.6	2.57
	Sand												
585	Gray	115.0	IL	335	0.48	120.5	1.05	1.00	120.8	1.05	0.83	134.6	1.17
	Fine	115.0	±5%	335	0.55	138.0	1.20	1.19	164.4	1.43	0.83	182.8	1.59
	Silty	115.0	±10%	335	0.61	153.0	1.33	1.38	212.8	1.85	0.83	236.9	2.06
	Sand												
570	Gray	375.9	IL	1340	0.48	482.0	1.34	1.00	493.1	1.34	0.65	434.2	1.18
	Fine	375.9	±5%	1340	0.55	552.0	1.50	1.19	666.1	1.81	0.65	588.8	1.60
	Sand	375.9	±10%	1340	0.61	613.0	1.67	1.38	853.8	2.32	0.65	754.4	2.05

TABLE 2.5-51

FACTOR OF SAFETY (FS) AGAINST LIQUEFACTION FOR AVERAGE RELATIVE DENSITY CONDITIONS

Level Ground at Elevation 584.0 ft

(1) Elev	(2) Soil	(3) τ_d (psf)	(4) Strain Condition	(5) σ_v (psf)	(6) $\left(\frac{\sigma_d}{2\sigma_{3c}}\right)$ N=10	(7) τ_f (psf)	(8) FS	(9) D_c (based on D_r)	(10) τ_f (psf)	(11) FS (C_r based on D_r)	(12) C_r (based on K_o)	(13) τ_f (psf)	(14) FS (C_r based on K_o)
582	Gray	48.2	IL	134	0.54	54.3	1.12	1.03	55.9	1.16	0.83	61.9	1.28
	Fine	48.2	±5%	134	0.63	63.3	1.31	1.25	79.1	1.64	0.83	87.6	1.82
	Sand	48.2	±10%	134	0.75	75.4	1.56	1.42	112.4	2.33	0.83	124.4	2.58
579	Gray	115.0	IL	335	0.54	135.7	1.18	1.03	139.7	1.21	0.83	154.6	1.34
	Fine	115.0	±5%	335	0.63	158.3	1.38	1.25	197.9	1.72	0.83	219.0	1.90
	Sand	115.0	±10%	335	0.75	188.4	1.64	1.42	267.6	2.33	0.83	296.1	2.58
577.5	Gray	148.8	IL	435.5	0.54	176.4	1.18	1.03	181.7	1.22	0.83	201.5	1.35
	Fine	148.8	±5%	435.5	0.63	205.8	1.38	1.25	257.2	1.73	0.83	285.3	1.92
	Sand	148.8	±10%	435.5	0.75	245.0	1.65	1.42	347.8	2.34	0.83	385.9	2.59
570	Gray	309.0	IL	938	0.54	379.9	1.23	1.03	391.3	1.27	0.73	380.8	1.23
	Fine	309.0	±5%	938	0.63	443.2	1.43	1.25	554.0	1.79	0.73	539.2	1.74
	Sand	309.0	±10%	938	0.75	527.6	1.71	1.42	749.2	2.42	0.73	729.2	2.36

TABLE 2.5-52

FACTOR OF SAFETY (FS) AGAINST LIQUEFACTION FOR LOW AVERAGE RELATIVE DENSITY CONDITIONS

(Level Ground at Elevation 584.0 ft)

(1) Elev	(2) Soil	(3) τ_d (psf)	(4) Strain Condition	(5) σ_v (psf)	(6) $\left(\frac{\sigma_d}{2\sigma_{3c}}\right)$ N=10	(7) τ_f (psf)	(8) FS	(9) D_c (based on D_r)	(10) τ_f (psf)	(11) FS (C_r based on D_r)	(12) C_r (based on K_o)	(13) τ_f (psf)	(14) FS (C_r based on K_o)
582	Gray	48.2	IL	134	0.48	48.2	1.00	1.00	48.2	1.00	0.83	53.5	1.11
	Fine	48.2	±5%	134	0.55	55.3	1.15	1.19	65.8	1.36	0.83	73.0	1.51
	Sand	48.2	±10%	134	0.61	61.3	1.27	1.38	84.6	1.76	0.83	93.8	1.95
579	Gray	115.0	IL	335	0.48	120.6	1.05	1.00	120.6	1.05	0.83	133.8	1.16
	Fine	115.0	±5%	335	0.55	138.2	1.20	1.19	164.4	1.43	0.83	182.4	1.59
	Sand	115.0	±10%	335	0.61	153.3	1.33	1.38	211.5	1.84	0.83	234.6	2.04
577.5	Gray	148.8	IL	435.5	0.48	156.8	1.05	1.00	156.8	1.05	0.83	173.9	1.17
	Fine	148.8	±5%	435.5	0.55	179.6	1.21	1.19	213.8	1.44	0.83	237.1	1.59
	Sand	148.8	±10%	435.5	0.61	199.2	1.34	1.38	274.9	1.85	0.83	305.0	2.05
570	Gray	309.0	IL	938	0.48	337.7	1.09	1.00	337.7	1.09	0.73	328.7	1.06
	Fine	309.0	±5%	938	0.55	386.9	1.25	1.19	460.4	1.49	0.73	448.2	1.45
	Sand	309.0	±10%	938	0.61	429.1	1.39	1.38	592.2	1.92	0.73	576.4	1.86

TABLE 2.5-53

SUMMARY OF STATIC AND DYNAMIC STABILITY ANALYSES FOR INTERIOR DIKE

LOADING CONDITIONS	MINIMUM FACTOR OF SAFETY PROVIDED
a. Static Loading conditions	
1. End of Construction - no water	2.2
2. Full Reservoir - Water Elevation 595 feet	2.0
3. Rapid Drawdown - Water Reduced from Elevation 595 feet to 592 feet	1.8
b. Pseudostatic Loading Conditions with 0.12 Seismic Coefficient	
1. End of Construction - no water	1.5
2. Full Reservoir - Water Elevation 595 feet	1.3
3. Rapid Drawdown - Water Reduced from Elevation 595 feet to 592 feet	1.2

Attachments 2.5A through 2.5D have been deleted intentionally.