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CHAPTER 2 - SITE CHARACTERISTICS

DRAWINGS CITED IN THIS CHAPTER*

* 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 USAR. They are controlled by the Controlled Documents Program.

DRAWING*	SUBJECT
M01-1101	Site Development
M01-1102	Site Development
M01-1103	Site Development
M01-1116	General Arrangement - Circulating Water Screen House
S03-1045	Site Development Plan Sh 5
S03-1100 through S03-1110	Site Grading and Drainage Plan Plant Area

CHAPTER 2 - SITE CHARACTERISTICS

2.0 SITE CHARACTERISTICS

This chapter provides information on the geological, seismological, hydrological, and meteorological characteristics of the Clinton Power Station (CPS) site and vicinity, in conjunction with present and projected population distribution and land use and site activities and controls. The purpose is to indicate how these site characteristics influenced plant design and operating criteria and to show the adequacy of the site characteristics from a safety viewpoint.

2.1 GEOGRAPHY AND DEMOGRAPHY

2.1.1 Site Location and Description

2.1.1.1 Specification of Location

The Clinton Power Station lies within Zone 16 of the Universal Transverse Mercator Coordinates with the approximate location of the reactor at coordinates 4,448,375 meters north and 343,825 meters east. This point is at 40° 10' 19.5" north latitude and 88° 50' 3" west longitude. Figure 2.1-1 shows the latitudinal and longitudinal location of the station.

Clinton Power Station with its associated man-made cooling reservoir (Lake Clinton) is an irregular U-shaped site in DeWitt County in east-central Illinois about 6 miles east of the city of Clinton. The station is near the confluence of Salt Creek and North Fork of Salt Creek about 56 miles east of where Salt Creek joins the Sangamon River.

DeWitt County is approximately 60 miles northeast of Springfield, almost midway between the cities of Decatur to the south, Champaign to the east, Bloomington to the north, and Lincoln to the west.

DeWitt County is almost equidistant between St. Louis and Chicago. Figure 2.1-1 shows the approximate location of the site within the state of Illinois. Figure 2.1-2 shows the location of the site in DeWitt County and the population centers of the county.

The majority of the site is located in the eastern half of DeWitt County with the arms of the lake extending into the northeastern area of the county. The site is located within Townships 19, 20, and 21 North, Range 3 East of the Third Principal Meridian, Townships 19, 20, and 21 North, Range 4 East of the Third Principal Meridian, and Townships 20 and 21 North, Range 5 East of the Third Principal Meridian. The reactor is located in Harp Township but the size and irregular shape of the site place it in several political subdivisions. These are the townships of Harp, Wilson, Rutledge, DeWitt, Creek, Nixon, and Santa Anna.

The location of the site within these townships is portrayed in Figure 2.1-3.

Also indicated in Figure 2.1-3 is Weldon Springs State Park, which is the only park of substantial size in DeWitt County other than the Lake Clinton recreational facilities. The state park encompasses approximately 370 acres and contains a 28-acre lake that is the only sizable lake within 10 miles of the site.

2.1.1.2 Site Area Map

Figure 2.1-4 is a site index plat showing the Clinton Power Station property lines. The site outer boundary is the same as the station outer property lines as shown in Figure 2.1-4. The site and its environs consist primarily of the generating station, Lake Clinton, woodlands, pasture land, cultivated farmland, and the recreational areas.

The total area encompassed by the outer site boundary is about 14,182 acres, of which about 450 acres are not station property as identified by the 14 property exception areas listed in Table 2.1-1 and shown in Figure 2.1-4. The station property of approximately 13,730 acres is owned by Exelon. The balance of the site property (5.5 acres for canceled Unit 2 and 38.5 acres unassigned) is owned solely by Exelon. Except for CPS, the CPS Energy and Environmental Center, now vacant, and the site recreational facilities, there are no industrial, commercial, or institutional structures on the site property. Four residential structures on site property are leased by Exelon. These houses are shown in Drawing M01-1101-2. The only other uses of site property not related to electrical production or recreation are for agriculture and water supply. Approximately 1500 acres of leased agricultural land will remain in use. A water well located south of the station will be a primary source of water for the village of DeWitt. The well location is shown in Figure 2.4-45.

Drawing M01-1101-02 is a site development drawing showing the location of the reactor on the site property. The reactor is about 3 miles northeast of the confluence of Salt Creek and the North Fork of Salt Creek. The site includes an area that extends approximately 14 miles along Salt Creek and approximately 8 miles along the North Fork of Salt Creek.

Drawings M01-1102 and M01-1103 show the location and orientation of the principal station structures. Lake Clinton, formed by the dam constructed near the confluence of Salt Creek and the North Fork of Salt Creek, has a surface area at normal lake level (690 feet mean sea level) of approximately approximately 4900 acres with an average depth of about 15.6 feet. Lake Clinton is totally within the site property boundary. The station facilities and the 3.4-mile discharge flume occupy about 150 acres and 130 acres, respectively.

The balance of the site, except for the area around the Lake Clinton dam and spillways and land leased for agriculture, is developed for recreation. The recreational development of the site is described in Subsection 2.1.3 of the Clinton Power Station Environmental Report - Operating License Stage (CPS-ER(OLS)).

The boundary line of the station exclusion area (as defined in 10 CFR 100) is shown in Drawing M01-1101-02. The exclusion area is entirely within the station property and is the area encompassed by a circle of 975 meters radius centered on the station standby gas treatment system vent. Drawing M01-1101-02 shows the low population zone as defined in 10 CFR 100.

The Clinton Power Station exclusion area meets the requirements of 10 CFR 100.11(a). The engineered safety features maintain the integrity of the containment under postulated accident conditions and limit exposures at the exclusion area and low population zone boundaries to levels well within the guidelines of 10 CFR 100, or, for the accidents analyzed using Alternative Source Terms, the limits of 10 CFR 50.67.

Figures 2.1-2 through 2.1-4 show highway and railway networks that traverse or are adjacent to the site. The nearest major highways are State Highways 54, 10, and 48, all of which cross the site. Other major thoroughfares are U.S. Highway 51, located about 6 miles west of the plant, and Interstate Highway 74, located about 11 miles northeast of the plant. The nearest railroad is the Canadian National/Illinois Central (CNIC) Railroad, which crosses CPS property about 0.75 mile to the north of the reactor centerline. Railroads surrounding the site are depicted in Figures 2.1-5 and 2.1-6. Major transportation routes and pipelines surrounding the site are shown in Figure 2.1-6.

2.1.1.3 Boundaries for Establishing Effluent Release Limits

The boundary of the restricted area (as defined in 10 CFR 20) is shown in Figure 2.1-7. There are no residential quarters in the restricted area. The radiation dose limits given in 10 CFR 20.1301 and the concentration limits of radioactive material in effluents given in 10 CFR 20.1302 are met at the restricted area boundary.

Access to the restricted area shown in Figures 2.1-7 and 2.1-8 known as the Owner Controlled Area is restricted by a cyclone fence with "No Trespassing" signs posted at regular intervals.

The distance in meters from the normal gaseous effluent release point (i.e., the common station HVAC vent) to the nearest site boundary by compass sectors is shown in Drawing M01-1101-02. The guidelines for keeping the radiation exposures as low as is reasonably achievable (ALARA), as given in 10 CFR 50 Annex to Appendix I, are applied at the site boundary. The station gaseous and liquid effluent release points are shown in Drawings M01-1102 and M01-1103.

The liquid effluents from the station are discharged into Lake Clinton, the outfall of which joins the Sangamon River approximately 56 miles downstream. The Sangamon River joins the Illinois River approximately 80 miles west of the site. The closest sizeable lake is Lake Decatur, located approximately 20 miles south of the site. There is no plausible way for liquid effluents to get to Lake Decatur.

The liquid effluents from the station are discharged into Lake Clinton through the discharge flume to the unrestricted area. The routine gaseous effluents discharged from the common station HVAC vent are released to the unrestricted area at the boundary of the restricted area in all of the sectors. Solid radioactive material is shipped from the CPS site via truck or rail in special containers or casks.

Expected concentrations of radionuclides in effluents are given in Sections 11.2 and 11.3.

2.1.2 Exclusion Area Authority and Control

2.1.2.1 Authority

The exclusion area for the Clinton Power Station meets the requirements of 10 CFR 100.11(a). The boundary line of the station exclusion area is shown in Drawing M01-1101-02. The exclusion area is entirely within the station property and is the area encompassed by a circle of 975 meters radius centered on the station standby gas treatment system vent.

Exelon owns all of the property in the exclusion area with the exception of a right-of-way for the township road which traverses the exclusion area. This road (shown in Figure 2.1-4) provides access to privately-owned property which lies outside the exclusion area within the peninsula area between the Salt Creek finger and the North Fork of Salt Creek finger of Lake Clinton. In an emergency, Exelon together with the local law enforcement agency (DeWitt County Sheriff's Department) will control access via this road to the exclusion area. The property ownership and mineral rights provide Exelon the authority to determine all activities, including exclusion and removal of personnel and property from the exclusion area. IP purchased an 8-inch underground oil pipeline segment which transversed the exclusion area and was previously owned by Ashland Oil, Inc. This pipeline segment was capped and abandoned during CPS construction. The Shell Oil Company 14-inch petroleum products pipeline was relocated outside of the exclusion area to a minimum distance of 4650 feet from the plant.

The primary activities in the exclusion area are those associated with the generation and distribution of electricity by the Clinton Power Station. There are no residences in the exclusion area.

A private rail spur from the Canadian National/Illinois Central Railroad track, which is located to the north of the site boundary, was constructed to the station. With the exception of the single township road, there are no other public highways, waterways, or railroads that traverse the exclusion area.

2.1.2.2 Control of Activities Unrelated to Plant Operation

In those areas subject to radiation from Unit 1, Exelon provides surveillance and controls construction worker occupancy as appropriate. None of the land within the exclusion area is planned for public recreational use. A small area of the Lake Clinton cooling lake lies within the exclusion area and is barricaded from use for public recreation lake activities. Only those activities will be authorized which provide assurance, under appropriate limitations, that no significant hazards would result to the public health and safety.

2.1.2.3 Arrangement for Traffic Control

In an emergency, the DeWitt County Sheriff's Department as assisted by other law enforcement agencies, will provide area control, communication assistance, and handling of the public, should evacuation become necessary. The Sheriff's office located in Clinton, Illinois, will be notified of an emergency at the plant. There is radio and telephone communication between the plant and the Sheriff's office.

2.1.2.4 Abandonment or Relocation of Roads

Parts of two township roads, which were located in the exclusion area, have been vacated. One of these roads has been relocated while the other was abandoned. The abandoned roadways have no public access or usage and are under the complete control of CPS.

All abandonment and relocation proceedings have been completed including:

a. a preliminary hearing with Harp Township officials,

b. an inducement agreement filing by IP,

c. public notice of a hearing on this matter,

d. a public hearing,

e. a preliminary order issuance,

f. a final order issuance.

Public authorities who made the final determination at the legislative public hearings were Wendall Reinhart, Highway Commissioner, Harp Township, DeWitt County, and Beverly J. Willoughby, Harp Township Town Clerk.

2.1.3 Population Distribution

Bureau of Census MEDLIST (Master Enumeration District List) was used to determine the 1970 population distribution. The MEDLIST tapes list the 1970 populations of states, counties, townships, and enumeration districts. Enumeration districts are small- population statistical units, averaging 800 people. On the census tapes, the 1970 population and the coordinates of the population centroid of each enumeration district are given.

Since the populations of the enumeration districts are given for population centroids, which may be separated by several miles in sparsely populated areas, use of the MEDLIST tapes without modification can result in zero populations in annular sectors within which the population is not, in fact, zero. A smoothing technique was developed, therefore, which distributes the population of the enumeration districts over a finite area surrounding the population centroid.

The population projections for 1980, 1990, 2000, 2010, and 2020 were made using a modified ratio technique. The ratio technique has been used by professional demographers and is based on the knowledge that the populations of larger areas are more accurately predicted than those of smaller areas, and that the ratio of the population of a smaller area, such as a township, to the population of a larger area, such as a state, changes at a constant rate. To determine the rate of change in the ratio, a historical base period is required. The rate of change in the ratio during the base period is then projected linearly. The base period for the projections was 1960-1970.

The ratio technique was modified in such a manner that the change in ratio established during the base period was essentially maintained for a few years, but the change was then gradually decreased to zero and the ratio itself thus became constant after 20 years. This modification reflects the fact that the growth rate of the smaller area may differ significantly from that of the larger area 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.

The modified ratio technique was applied on the basis of the ratio of the townships to the state. The state population was projected geometrically using the growth of the state during the base period.

For greater accuracy in the 0- to 10-mile region, an onsite house count was conducted in 1981. The number of houses counted was multiplied by 3, the average number of people per house based on Census Bureau statistics for the area, to arrive at the population.

The future populations, based on the 1981 house counts, were also projected using the modified ratio technique.

2.1.3.1 Population Within 10 Miles

The population within 5 miles of the site is shown in Table 2.1-9. These statistics as well as the 5 to 10 mile statistics are based on an actual onsite house count conducted by the DeWitt

County Farm Bureau and Illinois Power Company in September 1981. The total 1990 population within 5 miles of the site was 867. As indicated by a population density of 11 people/mi^2, the area within 5 miles of the site is very sparsely populated. There are two small residential groupings within 5 miles of the site: one is DeWitt, approximate population of 122, located 2.5 miles ENE; and the other is Lane, estimated population of 133, located 3.5 miles SSW.

As indicated in Table 2.1-9 the population within 10 miles of the site is quite low. A large proportion of the population lives in the city of Clinton (1990 population 7437), 6 to 7 miles W and WSW of the site. Most of the surrounding area is rural, with a population density of 40 people/mi^2, for the area within 10 miles of the site.

The total 1990 population within a 10-mile radius of the station is 11,957. Projections indicate that this area is expected to experience a gradual increase in population through the year 2030 (Table 2.1-10). The projected population for the year 2030 is 15,045.

There are few cities within 10 miles of the site and no major population centers (cities with populations greater than 25,000). Table 2.1-2 includes all the communities within 50 miles along with their 1970 populations.

2.1.3.2 Population Between 10 and 50 Miles

The 1970 populations and projections to the year 2020 for the area within 10 to 50 miles of the site are shown in Figure 2.1-10. The total population within 50 miles is 720,998 and is expected to grow to 1,202,310 by 2020. More than 90% of the total 1970 population within 50 miles live outside a 20-mile radius. Figure 2.1-11 shows the locations of the major cities within 50 miles of the site, the population of these cities is included in Table 2.1-2.

The most heavily populated sector within 50 miles of the site is the SSW sector with a 1970 population of 119,510. The high population in this sector is due primarily to Decatur, Illinois, which is located between 20 and 30 miles from the site and had a 1990 population of 83,885. The NE sector, which is predominately rural, has the lowest 1970 population.

2.1.3.3 Transient Population

Weldon Springs State Park, located about 5.5 miles southwest of the site, had 488,982 visitors in 1976 (Illinois Department of Conservation, 1976). On Independence Day attendance may reach 11,000 for the fireworks display (Herzog, 1977). The 370-acre park, 28 acres of which is lake, offers facilities for fishing, boating, and hiking (Illinois Department of Conservation, 1976). The park is popular due to the lack of other facilities in the county. The 1990 attendance is given in Table 2.1-3.

The Clinton Country Club, 6.3 miles southwest, has a membership of approximately 210. During the golfing season, approximately 75 to 80 golfers may use the course on a weekend day. On a peak attendance day, up to 90 people may be using the facilities, which include the golf course, pool, tennis courts, and dining room.

In addition to the existing recreational facilities, Lake Clinton will in itself attract recreation seekers, as shown in Table 2.1-4.

Arrowhead Acres Camp, located about 6.5 miles southwest of the site, operates from April through October and has about 200 long-term campers during the season and up to 400 campers on a weekend.

The Little Galilee Christian Assembly Church Camp (located approximately 6 miles southwest of Clinton, off Route 51) operates a summer camp between June and September. About 2000 people attend during the season with about 40 children in attendance during each week. A peak attendance of approximately 300 people occurs on weekends when parents pick up and drop off children.

The nearest industry is Van Horn-DeWitt, located approximately 2.5 miles east-northeast of the plant. Most of the employees reside locally and are not considered transients. Table 2.2-2 lists all industries within 10 miles of the site along with their approximate number of employees and products.

Table 2.1-5 shows the enrollment and staff of all schools near the Clinton Power Station. Discussions with the administration offices indicate that most of the staff and students reside locally and thus are considered as permanent population (Nernuh,1977, and Administration Center, Wapella C.U. School District, 1977). Besides the schools, the only other public facility is the John Warner Hospital, with a 52-bed capacity and a staff of 119, located in the city of Clinton.

Recreation and educational facilities are the only significant source of transient population. Recreationists at the Lake Clinton Recreation Area are included in Table 2.1-6. The distribution of recreationists can be summarized as shown in Table 2.1-3. The average daily recreational user and peak daily recreational users of the Lake Clinton Recreation Area are shown on Tables 2.1-11 and 2.1-12, respectively.

2.1.3.4 Low Population Zone

Table 2.1-6 gives the projected population distribution for the low population zone (LPZ) and includes permanent residents and the estimated peak daily transients attracted by the onsite recreational facilities.

The low population zone (as defined in 10 CFR 100) shown in Drawing M01-1101-02 is the area immediately surrounding the exclusion area encompassed by a circle of 2.5 miles radius (4018 meters) centered on the station standby gas treatment system vent. The LPZ does not include the city of DeWitt as the LPZ is tangent to the city limit. There are no schools, hospitals, or institutions within the LPZ.

The LPZ was selected to provide reasonable probability that appropriate protective measures could be taken to assure compliance with the guidelines of 10 CFR 100, or, for the accidents analyzed using Alternative Source Terms, the limits of 10 CFR 50.67. The number and density of residents in the LPZ are low and this enables effective evacuation procedures to be followed in the event of a serious accident.

Figure 2.1-2 shows the highway network around the site. The roads and highways within the area will be the primary transportation routes for evacuation. Table 2.1-7 lists the facilities and institutions within 5 miles of the LPZ which may require special consideration in evaluating emergency plans.

2.1.3.5 Population Center

A population center is defined in 10 CFR 100 as a densely populated center where there are about 25,000 inhabitants or more. The closest such center is Decatur, Illinois, located approximately 22 miles south-southwest of the site, which had a 1990 population of 83,885. Table 2.1-2 shows the 1990 populations, distances and directions from the site of all cities, towns and villages within 50 miles of the site. Figure 2.1-11 shows major population centers within 50 miles of the Clinton Power Station site, which are included in Table 2.1-2.

Farmer City, located approximately 11 miles to the east-northeast from the station is the largest city (1990 population 2114) adjacent to the site boundary. The city of Clinton, approximately 6 miles to the west, is the closest sizable city in the nearby area (1990 population 7437). DeWitt is a small village (1990 population 122) located approximately 3 miles to the east-northeast from the station site.

2.1.3.6 Population Density

The cumulative population of the projected 1982 and 2020 population is plotted on Figure 2.1-12. Also shown are two standard curves based on 500 people/mi^2 and 1,000 people/mi^2 for comparison. Table 2.1-8 shows the cumulative 1970 as well as the cumulative projected 1980 and 2020 population plotted in Figure 2.1-12. The 1970 population of 1199 for the area within a 5-mile radius of the plant dropped to 867 by 1990. The population then is expected to decrease to 740 by the year 2020.

The average 1970 population density within 50 miles of the site is 91.8 people/mi^2. This density is expected to grow to an average of 153 people/mi^2 by the year 2020. The average population density, within 5 miles of the site, was 10.9 in 1980 and is expected to decrease to 9.4 by the year 2020.

TABLE 2.1-1
ACREAGE OF CLINTON POWER STATION

EXCEPTION AREA	USAGE	ACREAGE	
Total Acreage of Clinton Power Station			13,586.46
1	Residence	13.92	
2	Unimproved	5.87	
3	Illinois Power Property (including the Farmer City Lift Pump Station and the Residence near Farmer City)(Parcel E, Exhibit B, Easement and License Agreement)	295.60	
4	Unimproved	10.34	
5	State Highway	1.67	
6	Railroad	19.74	
7	Buckeye Pipeline Pumping Station	23.30	
8	Cemetery (Lisenby)	7.66	
9	State Highway	3.14	
10	State Highway	2.02	
11	Cemetery	3.02	
12	State Highway	3.22	
13	Illinois Power Property (Farmer City Farm) (Parcel F, Exhibit B, Easement and License Agreement)	44.90	
14	Illinois Power Property (Township #238) (Parcel G, Exhibit B, Easement and License Agreement)	21.80	
15	Clinton Lake Marina	138.06	
	Total Area Excluded	594.26	594.26
Total Acreage Within Outer Site Boundary			14,180.74

NOTE: UNIT 2 HAS BEEN CANCELLED.

TABLE 2.1-2
CITIES, TOWNS AND VILLAGES WITHIN 50 MILES OF CLINTON POWER STATION

CITY OR TOWN	COUNTY	1980 POPULATION	1990 POPULATION	DISTANCE AND DIRECTION FROM THE SITE
DeWitt	DeWitt	232	122	2.5 miles ENE
Weldon	DeWitt	531	361	5.3 miles ESE
Clinton	DeWitt	8,014	7,437	6.3 miles W
Wapella	DeWitt	768	608	7.4 miles WNW
Deland	Piatt	509	458	10.1 miles ESE
Maroa	Macon	1,760	1,602	10.7 miles SW
Farmer City	DeWitt	2,252	2,114	11.2 miles ENE
Cisco	Piatt	333	282	11.7 miles SSE
Heyworth	McLean	1,598	1,627	12.3 miles NW
Argenta	Macon	994	940	12.4 miles S
LeRoy	McLean	2,870	2,777	13.1 miles NNE
Kenney	DeWitt	443	390	13.6 miles WSW
Oreana	Macon	999	847	15.6 miles S
Downs	McLean	561	620	15.7 miles N
Waynesville	DeWitt	569	440	15.7 miles WNW
Monticello	Piatt	4,753	4,549	16.6 miles SE
Mansfield	Piatt	921	929	17.0 miles E
Forsyth	Macon	1,072	1,275	17.0 miles SSW
Cerro Gordo	Piatt	1,553	1,436	19.6 miles SSE
Warrensburg	Macon	1,372	1,274	19.8 miles SW
McLean	McLean	836	797	19.9 miles WNW
Bellflower	McLean	421	405	19.9 miles NE
Ellsworth	McLean	244	224	20.2 miles NNE
Atlanta	Logan	1,807	1,616	21.3 miles WNW
Bement	Piatt	1,770	1,668	21.4 miles SE
Latham	Logan	564	482	21.5 miles SW
Arrowsmith	McLean	292	313	21.9 miles NNE
Mahomet	Champaign	1,986	3,103	22.1 miles E
Decatur	Macon	94,081	83,885	22.4 miles SSW
Bloomington	McLean	44,189	51,972	22.7 miles NNW
Saybrook	McLean	882	767	23.8 miles NE
Ivesdale	Champaign	339	339	24.5 miles SE
Normal	McLean	35,672	40,023	24.6 miles NNW
Foosland	Champaign	153	132	24.7 miles ENE
Mount Pulaski	Logan	1,783	1,610	25.3 miles WSW
Cooksville	McLean	259	211	26.3 miles NNE
Mount Zion	Macon	4,563	4,522	26.5 miles S
Fisher	Champaign	1,572	1,526	26.8 miles ENE
Stanford	McLean	720	620	26.8 miles NW
Armington	Tazewell	292	348	27.0 miles WNW
Lincoln	Logan	16,327	15,418	27.1 miles W
Niantic	Macon	761	647	27.2 miles SW
Towanda	McLean	630	856	27.2 miles N
Hammond	Piatt	556	527	28.1 miles SSE
Sadorus	Champaign	435	469	28.6 miles ESE

TABLE 2.1-2 (Cont'd)
CITIES, TOWNS AND VILLAGES WITHIN 50 MILES OF CLINTON POWER STATION

CITY OR TOWN	COUNTY	1980 POPULATION	1990 POPULATION	DISTANCE AND DIRECTION FROM THE SITE
Colfax	McLean	920	854	29.4 miles NNE
Illiopolis	Sangamon	1,118	934	29.8 miles SW
Champaign	Champaign	58,133	63,502	29.9 miles E
Danvers	McLean	921	981	30.2 miles NW
Minier	Tazewell	1,261	1,155	30.3 miles NW
Savoy	Champaign	2,126	2,674	30.7 miles ESE
Dalton City	Mountrie	574	573	30.8 miles S
Hudson	McLean	929	1,006	30.9 miles NNW
Gibson City	Ford	3,498	3,396	31.0 miles NE
Anchor	McLean	192	178	31.2 miles NNE
Atwood	Douglas/Piatt	1,464	1,253	31.3 miles SE
Hartsburg	Logan	379	306	31.5 miles W
Tolono	Champaign	2,434	2,605	31.7 miles ESE
Broadwell	Logan	183	146	31.7 miles WSW
Carlock	McLean	410	418	32.0 miles NNW
Urbana	Champaign	35,978	36,344	32.2 miles E
Macon	Macon	1,300	1,282	32.2 miles SSW
Lovington	Moultrie	1,313	1,143	32.4 miles SSE
Lexington	McLean	1,806	1,809	32.4 miles N
Garrett	Douglas	205	169	32.7 miles SE
Pesotum	Champaign	651	558	33.5 miles ESE
Thomasboro	Champaign	1,242	1,250	33.5 miles E
Hopedale	Tazewell	913	805	34.2 miles WNW
Emden	Logan	527	459	34.3 miles WNW
Elkhart	Logan	493	475	34.5 miles WSW
Mount Auburn	Christian	598	544	34.8 miles SW
Blue Mound	Macon	1,338	1,161	35.0 miles SSW
Elliott	Ford	370	309	35.2 miles NE
Kappa	Woodford	170	134	35.8 miles NNW
Arthur	Douglas/Moultrie	2,122	2,112	35.9 miles SSE
Bethany	Moultrie	1,550	1,369	36.0 miles S
Congerville	Woodford	373	397	36.1 miles NNW
Buffalo	Sangamon	514	503	36.3 miles SW
Philo	Champaign	973	1,028	36.3 miles ESE
Mackinaw	Tazewell	1,354	1,331	36.5 miles NW
Rantoul	Champaign	20,161	17,212	36.6 miles ENE
Sibley	Ford	370	359	36.9 miles NE
Tuscola	Douglas	3,839	4,155	37.6 miles SE
Mechanicsburg	Sangamon	515	538	37.7 miles SW
Moweaqua	Shelby	1,922	1,785	38.0 miles SSW
New Holland	Logan	295	330	38.1 miles W
Dawson	Sangamon	532	536	38.2 miles WSW
Delavan	Tazewell	1,973	1,642	38.8 miles WNW
Goodfield	Woodford	500	454	38.8 miles NNW
Middletown	Logan	503	436	38.8 miles W
Williamsville	Sangamon	996	1,140	39.2 miles WSW

TABLE 2.1-2 (Cont'd)
CITIES, TOWNS AND VILLAGES WITHIN 50 MILES OF CLINTON POWER STATION

CITY OR TOWN	COUNTY	1980 POPULATION	1990 POPULATION	DISTANCE AND DIRECTION FROM THE SITE
Gridley	McLean	1,246	1,304	39.2 miles N
Ludlow	Champaign	397	323	39.3 miles ENE
Chenoa	McLean	1,847	1,732	39.8 miles N
El Paso	Woodford	2,676	2,499	40.1 miles NNW
Villa Grove	Douglas	2,707	2,734	40.2 miles ESE
Sullivan	Moultrie	4,526	4,354	40.2 miles SSE
Stonington	Christian	1,184	1,006	40.3 miles SSW
Sidney	Champaign	886	1,027	40.3 miles ESE
Deer Creek	Tazewell	688	630	40.3 miles NW
San Jose	Logan/Mason	784	519	40.4 miles WNW
Melvin	Ford	519	466	40.6 miles NE
St. Joseph	Champaign	1,900	2,052	40.7 miles E
Spaulding	Sangamon	428	440	41.4 miles WSW
Tremont	Tazewell	2,096	2,088	41.4 miles NW
Riverton	Sangamon	2,783	2,638	42.0 miles WSW
Secor	Christian	488	389	42.1 miles NNW
Camargo	Douglas	428	372	42.3 miles SE
Arcola	Douglas	2,714	2,678	42.4 miles SE
Paxton	Ford	4,258	4,289	42.7 miles ENE
Fairbury	Livingston	3,544	3,643	42.7 miles NNE
Gifford	Champaign	848	845	42.7 miles ENE
Strawn	Livingston	143	132	42.7 miles NNE
Panola	Coles	31	43	43.0 miles NNW
Sherman	Sangamon	1,501	2,080	43.5 miles WSW
Eureka	Woodford	4,306	4,435	43.6 miles NNW
Morton	Tazewell	14,178	13,799	43.7 miles NW
Longview	Champaign	207	180	43.8 miles ESE
Mason City	Mason	2,719	2,323	44.0 miles W
Findlay	Shelby	868	787	44.1 miles S
Royal	Champaign	274	217	44.1 miles E
Allenville	Moultrie	204	166	44.2 miles SSE
Green Valley	Tazewell	768	745	44.3 miles WNW
Clear Lake	Sangamon	236	193	44.4 miles WSW
Edinburg	Christian	1,231	982	44.6 miles SW
Roberts	Ford	422	397	45.1 miles NE
Forrest	Livingston	1,246	1,124	45.2 miles NNE
Ogden	Champaign	818	671	45.2 miles E
Assumption	Christian	1,283	1,244	45.3 miles SSW
Loda	Iroquois	486	390	45.3 miles ENE
Cantrall	Sangamon	141	123	45.7 miles WSW
Homer	Champaign	1,279	1,264	44.8 miles ESE
Rochester	Sangamon	2,488	2,676	45.9 miles SW
Greenview	Menard	830	848	46.3 miles W
Broadlands	Champaign	346	340	46.4 miles ESE
Humboldt	Coles	499	470	46.4 miles SE
Grandview	Sangamon	1,794	1,647	46.4 miles WSW

TABLE 2.1-2 (Cont'd)
CITIES, TOWNS AND VILLAGES WITHIN 50 MILES OF CLINTON POWER STATION

CITY OR TOWN	COUNTY	1980 POPULATION	1990 POPULATION	DISTANCE AND DIRECTION FROM THE SITE
Roanoke	Woodford	2,001	1,910	46.7 miles NNW
Washington	Tazewell	10,364	10,099	46.8 miles NW
South Pekin	Tazewell	1,243	1,184	47.2 miles WNW
Athens	Menard	1,371	1,404	47.4 miles WSW
Taylorville	Christian	11,386	11,133	48.1 miles SSW
Flanagan	Livingston	978	987	48.3 miles N
Hindsboro	Douglas	407	346	48.6 miles SE
Springfield	Sangamon	99,637	105,227	48.6 miles WSW
Benson	Woodford	460	410	48.7 miles NNW
Chatsworth	Livingston	1,187	1,186	48.7 miles NE
Pekin	Tazewell	33,967	32,254	49.0 miles NW
Kincaid	Christian	1,591	1,353	49.1 miles SW
Allerton	Vermillion	303	274	49.1 miles ESE
Fithian	Vermillion	540	512	49.1 miles E
Bulpitt	Christian	301	206	49.3 miles SW
Pontiac	Livingston	11,227	11,428	49.5 miles NNE
East Peoria	Tazewell	22,385	21,378	49.5 miles NW
Jeiseyville	Christian	178	126	49.5 miles SW
Marquette Heights	Tazewell	3,386	3,077	49.8 miles NW
Owaneco	Christian	285	260	49.9 miles SSW

Source: 1980 - U.S. Department of Commerce, 1981.
1990 - Illinois Counties and Incorporated Municipalities.

TABLE 2.1-3
RECREATIONAL FACILITIES IN THE VICINITY
OF CLINTON POWER STATION

AREA	ATTENDANCE AVERAGE DAILY	ATTENDANCE PEAK DAILY	APPROXIMATE DISTANCE AND DIRECTION FROM THE STATION
Planned Onsite Facilities	1,050*	10,309**	
Arrowhead Acres	200	400	6.5 miles SW
Clinton Country Club	50	90	8.0 miles SW
Little Galilee Christian Assembly Church Camp	140	140	10.5 miles SW
Weldon Springs State Park	1,200	4,200	5.5 miles SW
Calvary United Church Camp	111	550	9.0 miles NW
Green Acres Campground	77	150	9.0 miles W
TOTAL	2,828	15,839	

* From Table 2.1-11
** From Table 2.1-4

Sources: 1. Illinois Power Company, 1981; Ferguson, 1977; Hoffman, 1978; Johnson, 1977; and Herzog, 1977.

2. Evacuation Time Estimates for the Clinton Power Station Plume Exposure Pathway Emergency Planning Zone.

TABLE 2.1-4
PEAK DAY USERS OF CLINTON LAKE RECREATIONAL AREA
BASED ON AVAILABLE PARKING CAPACITY, 1990

FACILITY NAME	CAR	CAR TRAILER	CAMP SITE	PEAK DAY RECREATIONAL USERS
Parnell	5	17	-	50[2]
Weldon	39	250	-	750
Northfork	15	84	-	250
Westside	63	250	-	750
Camp Quest	334	3[1]	-	1000
Visitors Center	50	3[1]	-	150
Mascoutin	1,334	40	348	4,000
Marina	255	1,000	-	3,000
Department of Conservation Hdqrs.	18	4	-	108
Lane	20	17	-	50
Penninsula	25	4	0	10
Tail Water	36	-	-	176
Northfork Canoe	20	5	0	15
TOTAL	2,214	1,677	348	10,309[3]

[1]Area designed for bus parking - 180 passengers per bus.

[2]Calculated by formula: Number of parking spaces x 3.5 x 1.4, where 3.5 equals average number of passengers per car and 1.4 equals 40% overflow parking capacity.

[3]Recreational Area was designed for about 4700 users.

Source: Evacuation Time Estimates for the Clinton Power Station Plume Exposure Pathway Emergency Planning Zone.

TABLE 2.1-5
SCHOOLS WITHIN 10 MILES OF CLINTON POWER STATION

SCHOOL	DISTANCE AND DIRECTION FROM THE SITE	ENROLLMENT		STAFF	
		1976-77[b]	1990[e]	1976-77[b]	1990[e]
Clinton					
Clinton High School	7.8 mi WSW	777	575	48	80
Clinton Junior High School	7.8 mi W	575	516	33	40
Douglas Grade School	6.7 mi W	226	260	12	35
Kenney Grade School	14.9 mi SW	---*		---*	
Kenney Kindergarten	14.9 mi SW	--*		---*	
Lincoln Grade School	7.8 mi WSW	174	264	09	17
Washington Grade School	7.8 mi W	382	289	20	35
Webster Grade School	6.7 mi W	230	204	17	23
Tiny Tot Nursery	7.8 mi WSW		111		16
Creative Corners Nursery	6.7 mi W		19		2
Head Start	6.7 mi W		24		4
Wapella					
Wapella Junior High and High School[c]	7.4 mi WNW	180	120	16	16
Wapella Grade School	7.8 mi WNW	182	126	13	14
Weldon					
Deland Weldon High School	7.8 mi ESE		71		15
DeLand Weldon Grade School[d]	7.8 mi ESE	281**	180	13**	20

* School has been closed.
**1975-1976

Source:
- b. Nernuh, 1977.
- c. Administration Center, Wapella C.U. School District, 1977.
- d. Illinois State Board of Education, 1970.
- e. Evacuation Time Estimates for the Clinton Power Station Plume Exposure Pathway Emergency Planning Zone.

TABLE 2.1-6
POPULATION DISTRIBUTION WITHIN THE LOW POPULATION ZONE OF CLINTON POWER STATION

SECTOR DESIGNATION	PROJECTED POPULATION DISTRIBUTION (PERMANENT + TRANSIENTS)					
	1981	1990	2000	2010	2020	2030
N	183 (3+180)	181 (1+180)	181 (1+180)	181 (1+180)	181 (1+180)	181 (1+180)
NNE	13 (13+0)	6 (6+0)	4 (4+0)	5 (5+0)	5 (5+0)	5 (5+0)
NE	3 (3+0)	1 (1+0)	1 (1+0)	1 (1+0)	1 (1+0)	1 (1+0)
ENE	34 (34+0)	31 (31+0)	31 (31+0)	32 (32+0)	33 (33+0)	34 (34+0)
E	30 (30+0)	22 (22+0)	21 (21+0)	22 (22+0)	22 (22+0)	23 (23+0)
ESE	2734 (0+2734)	2734 (0+2734)	2734 (0+2734)	2734 (0+2734)	2734 (0+2734)	2734 (0+2734)
SE	2733 (0+2733)	2733 (0+2733)	2733 (0+2733)	2733 (0+2733)	2733 (0+2733)	2733 (0+2733)
SSE	1409 (0+1409)	1412 (3+1409)	1412 (3+1409)	1412 (3+1409)	1412 (3+1409)	1412 (3+1409)
S	219 (0+219)	219 (0+219)	219 (0+219)	219 (0+219)	219 (0+219)	219 (0+219)
SSW	53 (0+53)	53 (0+53)	53 (0+53)	53 (0+53)	53 (0+53)	53 (0+53)
SW	3 (3+0)	1 (1+0)	1 (1+0)	1 (1+0)	1 (1+0)	1 (1+0)
WSW	109 (3+106)	107 (1+106)	107 (1+106)	107 (1+106)	107 (1+106)	107 (1+106)
W	902	899	898	898	898	898

TABLE 2.1-6 (Cont'd)

SECTOR DESIGNATION	PROJECTED POPULATION DISTRIBUTION (PERMANENT + TRANSIENTS) 1981	1990	2000	2010	2020	2030
	(6+896)	(3+896)	(2+896)	(2+896)	(2+896)	(2+896)
WNW	117 (11+106)	111 (5+106)	110 (4+106)	110 (4+106)	110 (4+106)	110 (4+106)
NW	1169 (11+1158)	1163 (5+1158)	1162 (4+1158)	1162 (4+1158)	1162 (4+1158)	1162 (4+1158)
NNW	112 (6+106)	109 (3+106)	108 (2+106)	108 (2+106)	108 (2+106)	108 (2+106)
Sum for Radial Interval	9823 (123+9700)	9782 (82+9700)	9775 (75+9700)	9778 (78+9700)	9779 (79+9700)	9781 (81+9700)
Average Density (persons/mi^2) in Radial Region of:						
Permanent Residents	6.3	4.2	3.8	4.0	4.0	4.1
Total Including Peak Number of Transients	500.3	498.2	497.8	498.0	498.0	498.1

Transient population based on peak daily recreational use (Table 2.1-12) of Lake Clinton within the Low Population Zone.
Permanent residents based on actual house count information (Table 2.1-9).

TABLE 2.1-7
PUBLIC FACILITIES AND INSTITUTIONS WITHIN 5 MILES OF
THE LOW POPULATION ZONE OF CLINTON POWER STATION

FACILITY/INSTITUTION	TYPE	SECTOR	APPROXIMATE DISTANCE FROM LPZ (miles)	ESTIMATED DAILY POPULATION[a]
Clinton School District	Educational	W	3.8	
Clinton High School				655
Clinton Junior High School				556
Douglas Grade School				295
Lincoln Grade School				281
Washington Grade School				324
Webster Grade School				227
Wapella School District	Educational	WNW	4.9	
Wapella Junior High and High School				136
Wapella Grade School				140
Weldon School District	Educational	ESE	2.8	
Deland Weldon Grade School				200
Deland Weldon High School				86
John Warner Hospital[b]	Medical	W	3.8	74
Crest View Nursing Home[c]	Medical	W	3.8	131
DeWitt County Nursing Home[d]	Medical	W	3.8	113
DeWitt Country Mental Health[e] Center	Medical	W	3.8	50

TABLE 2.1-7 (Cont'd)

FACILITY/INSTITUTION	TYPE	SECTOR	APPROXIMATE DISTANCE FROM LPZ (miles)	ESTIMATED DAILY POPULATION[a]
DeWitt County Court House[f]	Local Government	W	3.8	74
Clinton Country Club	Recreational	SW	3.8	90
Arrowhead Acres Camp	Recreational	SW	4.0	200
Little Galilee Christian				
Assembly Church Camp	Recreational	SW	3.5	140
Weldon Springs State Park	Recreational	SW	3.0	1200

[a] Evacuation Time Estimates for the Clinton Power Station Plume Exposure Pathway Emergency Planning Zone.

[b] Debbie Hendricks, Administrator, John Warner Hospital.

[c] Douglas Graves, Administrator, Crest View Nursing Home.

[d] Sally Jones, DeWitt Country Nursing Home.

[e] Nancy Cook, DeWitt County Mental Health Center.

[f] Helen Curl, Circuit Clerk, DeWitt Country Court House.

TABLE 2.1-8
CUMULATIVE POPULATION DISTRIBUTION

RADIUS	1970 POPULATION	1980 POPULATION	2020 POPULATION	STANDARD CURVES BASED ON 500 PEOPLE/mi^2	STANDARD CURVES BASED ON 1000 PEOPLE/mi^2
0-5	1,199	828	1,036	39,270	78,540
0-10	13,143	12,976	18,608	157,080	314,159
0-20	52,824	57,912	85,404	628,319	1,256,637
0-30	312,605	352,250	522,943	1,413,717	2,827,433
0-40	499,889	566,238	841,626	2,513,274	5,026,548
0-50	720,998	810,223	1,202,310	3,926,991	7,853,981

TABLE 2.1-9
POPULATION WITHIN 10 MILES OF THE CLINTON POWER STATION - 1981

SECTOR	DISTANCE FROM SITE IN MILES 0-1	1-2	2-3	3-4	4-5	5-10	0-5	0-10
N	3	0	16	21	13	84	53	137
NNE	0	10	3	26	23	99	62	161
NE	0	3	0	3	10	99	16	115
ENE	0	0	177	0	13	196	190	386
E	8	3	58	0	8	70	77	147
ESE	0	0	0	13	5	101	18	119
SE	0	0	3	8	13	609	24	633
SSE	0	0	10	5	10	98	25	123
S	0	0	3	13	16	109	32	141
SSW	0	0	0	146	23	96	169	265
SW	3	0	0	13	10	234	26	260
WSW	0	3	5	5	23	5356	36	5392
W	0	3	16	5	8	3549	32	3581
WNW	0	3	8	8	13	875	32	907
NW	0	8	10	3	10	195	31	226
NNW	3	0	8	10	13	71	34	105
TOTAL	17	33	317	279	211	11841	857	12698

Source: Population estimated from onsite house counts conducted by the DeWitt County Farm Bureau and Illinois Power Company, September 1981.

TABLE 2.1-10
POPULATION WITHIN 10 MILES OF THE CLINTON* POWER STATION
1990

	DISTANCE FROM SITE IN MILES							
SECTOR	0-1	1-2	2-3	3-4	4-5	5-10	0-5	0-10
N	0	5	15	14	31	61	65	126
NNE	0	5	7	12	15	120	39	159
NE	0	4	7	10	10	90	31	121
ENE	1	44	123	24	15	145	207	352
E	0	2	14	6	20	64	42	106
ESE	1	0	0	8	4	79	9	88
SE	1	0	1	10	23	474	22	496
SSE	1	0	7	10	11	75	29	104
S	0	0	2	23	16	69	41	110
SSW	0	0	0	93	46	114	139	253
SW	0	0	20	27	16	271	62	333
WSW	0	0	8	16	21	6,069	45	6,114
W	0	1	5	12	16	2,382	34	2,426
WNW	0	2	7	10	10	796	29	825
NW	0	6	6	15	12	213	40	253
NNW	0	5	13	7	8	58	33	91
TOTAL	4	74	235	299	274	11,080	867	11,957

* Evacuation Time Estimates for the Clinton Power Station Plume Exposure Pathway Emergency Planning Zone.

TABLE 2.1-10 (cont'd)

2000

	DISTANCE FROM SITE IN MILES							
SECTOR	0-1	1-2	2-3	3-4	4-5	5-10	0-5	0-10
N	1	0	5	13	9	67	28	95
NNE	0	3	1	15	20	107	39	146
NE	0	1	0	3	10	102	14	116
ENE	0	0	185	0	14	224	199	423
E	3	1	61	0	8	77	73	150
ESE	0	0	0	14	5	103	19	122
SE	0	0	3	8	12	581	23	604
SSE	0	0	9	5	9	95	23	118
S	0	0	3	12	15	107	30	137
SSW	0	0	0	137	22	93	159	252
SW	1	0	0	12	9	284	22	306
WSW	0	1	2	2	8	6013	13	6026
W	0	1	5	2	3	3918	11	3929
WNW	0	1	3	3	11	1102	18	1120
NW	0	3	3	1	3	235	10	245
NNW	1	0	3	3	9	55	16	71
TOTAL	6	11	283	230	167	13163	697	13860

TABLE 2.1-10 (cont'd)

2010

SECTOR	DISTANCE FROM SITE IN MILES 0-1	1-2	2-3	3-4	4-5	5-10	0-5	0-10
N	1	0	6	14	10	68	31	99
NNE	0	4	1	16	20	110	41	151
NE	0	1	0	3	11	104	15	119
ENE	0	0	190	0	14	230	204	434
E	3	1	62	0	9	79	75	154
ESE	0	0	0	14	5	106	19	125
SE	0	0	3	8	13	597	24	621
SSE	0	0	10	5	10	98	25	123
S	0	0	3	13	15	110	31	141
SSW	0	0	0	141	22	96	163	259
SW	1	0	0	13	10	292	24	316
WSW	0	1	2	2	8	6180	13	6193
W	0	1	6	2	3	4027	12	4039
WNW	0	1	3	3	11	1133	18	1151
NW	0	3	4	1	4	242	12	254
NNW	1	0	3	4	10	57	18	75
TOTAL	6	12	293	239	175	13529	725	14254

TABLE 2.1-10 (cont'd)

2020

SECTOR	DISTANCE FROM SITE IN MILES 0-1	1-2	2-3	3-4	4-5	5-10	0-5	0-10
N	1	0	6	14	10	70	31	101
NNE	0	4	1	16	21	113	42	155
NE	0	1	0	3	11	107	15	122
ENE	0	0	195	0	14	237	209	446
E	3	1	64	0	9	81	77	158
ESE	0	0	0	14	6	109	20	129
SE	0	0	3	8	13	613	24	637
SSE	0	0	10	5	10	101	25	126
S	0	0	3	13	16	114	32	146
SSW	0	0	0	145	23	98	168	266
SW	1	0	0	13	10	300	24	324
WSW	0	1	2	2	8	6351	13	6364
W	0	1	6	2	3	4139	12	4151
WNW	0	1	3	3	11	1164	18	1182
NW	0	3	4	1	4	249	12	261
NNW	1	0	3	4	10	58	18	76
TOTAL	6	12	300	243	179	13904	740	14644

TABLE 2.1-10 (cont'd)

2030

SECTOR	0-1	1-2	2-3	3-4	4-5	5-10	0-5	0-10
	DISTANCE FROM SITE IN MILES							
N	1	0	6	14	10	72	31	103
NNE	0	4	1	17	21	116	43	159
NE	0	1	0	3	11	110	15	125
ENE	0	0	201	0	15	243	216	459
E	3	1	66	0	9	83	79	162
ESE	0	0	0	15	6	112	21	133
SE	0	0	3	8	13	630	24	654
SSE	0	0	10	5	10	103	25	128
S	0	0	3	13	16	117	32	149
SSW	0	0	0	149	23	101	172	273
SW	1	0	0	13	10	309	24	333
WSW	0	1	2	2	9	6527	14	6541
W	0	1	6	2	3	4254	12	4266
WNW	0	1	3	3	11	1196	18	1214
NW	0	3	4	1	4	256	12	268
NNW	1	0	3	4	10	60	18	78
TOTAL	6	12	308	249	181	14289	756	15045

TABLE 2.1-11
AVERAGE RECREATIONAL USAGE
LAKE CLINTON RECREATION AREA

	DIRECTION IN MILES					
SECTOR	0-1	1-2	2-3	3-4	4-5	0-5
PLANT	1,050	0	0	0	0	1,050
N	0	0	5	0	0	5
NNE	0	0	0	0	0	0
NE	0	0	0	0	0	0
ENE	0	0	0	0	0	0
E	0	0	0	0	500	500
ESE	0	0	2,000	0	0	2,000
SE	0	0	0	0	0	0
SSE	0	1,500	0	0	0	1,500
S	0	0	0	0	0	0
SSW	0	0	0	0	0	0
SW	0	0	10	300	0	310
WSW	0	1	500	0	0	501
W	0	25	0	0	0	25
WNW	150	0	0	0	0	150
NW	0	0	0	0	0	0
NNW	0	150	0	0	0	150
TOTAL	150	1,675	2,515	300	500	4,086

Source: Evacuation Time Estimates for the Clinton Power Station Plume Exposure Pathway Emergency Planning Zone.

TABLE 2.1-12
PEAK DAILY RECREATIONAL USERS
LAKE CLINTON RECREATION AREA

	DIRECTION IN MILES					
SECTOR	0-1	1-2	2-3	3-4	4-5	0-5
PLANT	100	0	0	0	0	100
N	0	0	15	0	0	15
NNE	0	0	0	0	0	0
NE	0	0	0	0	0	0
ENE	0	0	0	0	0	0
E	0	0	0	0	750	750
ESE	0	0	4,000	0	0	4,000
SE	0	0	0	0	0	0
SSE	0	3,000	0	0	0	3,000
S	0	0	0	50	0	50
SSW	0	0	0	0	0	0
SW	0	0	10	500	0	510
WSW	0	0	750	0	0	750
W	0	1,000	0	0	0	1,000
WNW	0	0	0	0	0	0
NW	0	0	0	0	0	0
NNW	0	250	0	0	0	250
TOTAL	100	4,250	4,775	550	750	10,425

Source: Evacuation Time Estimates for the Clinton Power Station Plume Exposure Pathway Emergency Planning Zone.

TABLE 2.1-13
EPZ POPULATION CHANGE REPORT
CLINTON STATION – 1990/2000 POPULATION

Location	Census 1990	Census 2000	Gain/Loss	% Change
DeWitt County	16,516	16,798	282	1.7
Clintonia	7,860	7,926	66	0.8
Creek	412	402	-10	-2.4
DeWitt	417	465	48	11.5
Harp	250	335	85	34.0
Nixon	579	590	11	1.9
Rutledge	189	201	12	6.3
Santa Anna	2,550	2,487	-63	-2.5
Texas	1,028	1,284	256	24.9
Wapella	1,031	1,004	-27	-2.6
Wilson	150	155	5	3.3
DeWitt County EPZ Totals	14,466	14,849	383	2.6
Piatt	15,548	16,365	817	5.3
Blue Ridge	1,407	1,414	7	0.5
Goose Creek	848	852	4	0.5
Piatt County EPZ Totals	2,255	2,266	11	0.5
McLean	129,180	150,433	21,253	16.5
Downs	992	1,079	87	8.8
Empire	3,379	3,845	466	13.8
McLean County EPZ Totals	4,371	4,924	553	12.7
Macon	117,206	114,706	-2,500	-2.1
Friends Creek	1,429	1,456	27	1.9
Macon County EPZ Totals	1,429	1,456	27	1.9
Full EPZ Totals	22,521	23,495	974	4.3

2.2 NEARBY INDUSTRIAL, TRANSPORTATION, AND MILITARY FACILITIES

2.2.1 Locations and Routes

The highway transportation and routes within 5 miles of the Clinton Power Station (CPS) are shown in Figure 2.2-1. The nearest highways to the station are Illinois State Routes 54, 48, and 10 and U.S. Highway 51. Illinois State Route 54 passes through CPS site property approximately 3/4 mile north of the power station. Illinois State Route 10 is adjacent to the station, and Illinois State Route 48 is approximately 4.5 miles east of the station. Figure 2.2-1 illustrates these highways and their traffic volumes. U.S. Highway 51, located approximately 6 miles west of the CPS site is the most heavily travelled highway in the vicinity averaging approximately 13,000 cars per day. State Routes 10 and 54 are moderately well travelled having a 24-hour annual average of over 2000 cars within 5 miles of the station. The traffic volume within a 50-mile radius of the site is shown in Figure 2.2-1.

As shown in Figure 2.1-6, there is one railroad within 5 miles of the Clinton Power Station. The Canadian National/Illinois Central Railroad runs parallel to State Route 54 and traverses the property approximately 3/4 mile north of the power station.

There are three Illinois National Guard units within about fifty miles of the plant. There is an Army National Guard unit at the Decatur Airport (24 miles south). There is an Air National Guard unit at the Springfield Airport (52 miles southwest). And there is another Air National Guard unit at Peoria Airport (58 miles northwest).

The several military reserve unit armories located in the general area of the site are listed in Table 2.2-1. These armories normally should contain no explosives.

There are no military missile sites within 50 miles of the station.

The nearest industry to the station site is located approximately 2.5 miles to the east-northeast. Table 2.2-2 provides a listing of the industries within 10 miles of Clinton Power Station along with their respective products.

Airports and Low Altitude Federal Airways within 25 miles are shown in Figure 2.2-3. The Clinton Power Station site lies midway between the Decatur and the Bloomington-Normal Airports, which are located approximately 22.5 miles north and south respectively. These are the closest airports with commercial traffic. The locations and layouts of airports within 5 miles of CPS, and Decatur and Bloomington airports are illustrated in Figure 2.2-4.

Pipelines within 5 miles of the Clinton Power Station are shown in Figure 2.1-6 and are listed in Table 2.2-4.

2.2.2 Descriptions

2.2.2.1 Description of Facilities

Industries within 5 miles of the Clinton Power Station are listed in Table 2.2-2 along with their respective products, and approximate employment. As shown in Table 2.2-2 the area within 5 miles of the site is not heavily industrialized. The nearest industry is located approximately 2.5 miles east-northeast of the station.

2.2.2.2 Description of Products and Materials

Table 2.2-5 identifies the locations of the potentially hazardous chemicals and the quantities stored. Chemicals stored at the Clinton Power Station in quantities of significant bulk are listed in Table 2.2-6.

There are only two industries within 5 miles of the station which regularly manufacture, store, or use any type of hazardous material. Van Horn - DeWitt stores agricultural chemicals such as herbicides and insecticides as well as fertilizers and Evergreen FS maintains a large propane tank at their facility in DeWitt. In addition, Weldon Fertilizer and Lumber, Inc., and Evergreen FS, Inc.-Wapella also store hazardous materials which could impact the station.

Compliance with Regulatory Guide 1.78 for hazardous chemicals stored onsite is described in Section 6.4.

2.2.2.3 Pipelines

There are five pipelines that cross site property. During construction, discussions were held with the companies which operate the pipelines resulting in rerouting of pipelines, monitoring programs, and methods of controlling leakage onsite.

One pipeline traverses land near the station exclusion area. It is a 14-inch diameter pipeline owned and operated by Buckeye Partners, L.P. located 4650 feet from the station at the closest point. The pipeline is currently used for transporting refined petroleum products such as gasoline, fuel oil, and liquid petroleum gas. This pipeline, relocated during construction, has been used for transporting propane and butane in the past (the last movement was May 30, 1973), however, discussions with Buckeye Partners, L.P. have shown that future movements of propane and butane are unlikely. In the event that propane or butane would be transported in this pipeline, an agreement has been made for advance notice to Clinton Power Station and establishment of safety measures.

The safety measures to be taken in the event that Buckeye Partners, L.P. does ship propane or butane through this pipeline are delineated in a formal agreement. The following is an excerpt of safety measures from that agreement:

"III-l. VALVES - At all times during the operation of new Segments A and B, Buckeye Partners, L.P. shall maintain automatic valves in its DeWitt pumping station which, when operated in conjunction with the automatic gate valve at the north end of new Segment A and the automatic gate valve at the south end of new Segment B, will permit the isolation of either or both of the segments.

III-2. PROPANE & BUTANE SHIPMENTS - In the event that Buckeye Partners, L.P. ships propane or butane through new Segment A of the pipeline after the Clinton Power Station is operational -

(a) Buckeye Partners, L.P. shall:

(1) Give CPS not less than 7 days advance notification of each such shipment; and

(2) At the request of CPS, station and maintain one of its employees at its DeWitt pumping station, at all times during each such shipment, which employee (i) shall be in radio communication with CPS, employees engaged in patroling and monitoring new Segment A during each such shipment and (ii) shall have authority and orders, in the event of any abnormality in the new Segment A, during any such shipment of propane or butane, to close such valves as may be required in order to shut down and isolate new Segment A."

CPS will request the attendance of one of Buckeye Partners, L.P. employees at the DeWitt pumping station in accordance with paragraph III-2(a) (2) of this agreement as quoted above.

In addition to these safety measures, a hazard analysis was presented in the Preliminary Safety Analysis Report (PSAR) Appendix A2.2. The NRC request for additional information on this analysis and the subsequent agreement to move the pipeline to its present position of at least 4650 feet from the plant, are documented in PSAR Question 2.2-12, Volume 10, page 2-220. The NRC review and closure of this issue is documented in its Safety Evaluation Report for CPS Construction Permit NUREG-75/013 Sections 1.7a, 2.2.5, and 3.3 and Supplement No. 1 to NUREG-75/013 Section 2.2.5.

There are three other pipelines which cross the site property approximately 13,700 feet from the station. One is a 24-inch diameter line owned and operated by the Explorer Pipeline Company. The Conoco Phillips Pipe Line Company owns and operates two parallel 8-inch diameter pipelines. These three pipelines carry refined petroleum products similar to the Buckeye pipeline on a daily basis. Explorer and Conoco Phillips do not notify the plant of pipeline usage.

There is also a 2-inch natural gas pipeline owned by Ameren Illinois which passes within approximately 12,000 feet of the station.

All of the pipelines discussed above are listed in Table 2.2-4 which indicates the size, age, operating pressure, burial depth, and location and type of isolation valves of each line.

2.2.2.4 Waterways

The Clinton Power Station is located between the two fingers of an impoundment (Lake Clinton) created by the damming of Salt Creek and the North Fork of Salt Creek. There is no commercial traffic on Lake Clinton or on either creek. There is some recreational boating and fishing on Lake Clinton. The recreational facilities associated with the Clinton Power Station site, which include a marina, are described in Subsection 2.1.3 of the Clinton Power Station Environmental Report Operating License Stage.

2.2.2.5 Airports, Heliports and Airways

2.2.2.5.1 Airports and Heliports

Airports within 5 miles of CPS and Decatur and Bloomington airports are included in Figure 2.2-4.

A heliport is located on-site for use by company helicopters.

2.2.2.5.2 Airways

The closest Low Altitude Federal Airway to the site is V313 with its centerline passing within 2 miles east of the station. Other low altitude federal airways in the vicinity of the CPS site are V233 passing within 3 miles northwest, V72 with centerline approximately 5 miles northeast, and V434 with centerline approximately 6 miles north-northeast. These airways, used by aircraft operating below 18,000 feet (MSL) are shown in Figure 2.2-3, with respect to the station.

2.2.2.6 Projections of Industrial Growth

Industries within 10 miles of the station are listed in Table 2.2-2. There is no indication of any plans for expansion of these industries in the near future.

All airports within 5 miles are included in Figure 2.2-4. Because these are small private airports used for single engine planes, expansion of these airports is not likely in the near future. Table 2.2-4 lists the pipelines within 5 miles of the station. There is no planned expansion for any of these pipelines.

2.2.3 Evaluation of Potential Accidents

No nearby industrial or other activities have been identified which could pose unusual hazards to the Clinton Power Station.

The nearest highway is State Highway 54 which passes about three-quarters of a mile from the reactor containment building. U.S. Highway 51, is approximately 6 miles from the site. The nearest railroad is the Gilman Line of the Canadian National/Illinois Central Railroad which runs parallel to Highway 54 and traverses north of the site approximately .75 miles. Effects of accidents on these transportation routes have been evaluated and it is concluded that they need not be considered as design basis events. The station is not located near a navigable waterway. Airways hazards are discussed in Section 3.5.

Railroad transportation hazards were determined based upon existing patterns of hazardous material shipping for the Gilman Line of the Canadian National/Illinois Central Railroad. To ensure that future changes in these patterns do not affect the transportation hazards evaluation, a periodic survey of traffic on this line will be performed every 6 years. With regard to toxic materials, upon determination that acceptance levels have been exceeded, sensors will be installed that will isolate the control room heating,ventilation, and air conditioning system should the identified chemicals present a potential toxic environment.

2.2.3.1 Determination of Design Basis Events

The accident categories given below have been evaluated.

2.2.3.1.1 Explosions

Fluids, explosives, munitions, and chemicals stored or being transported in the vicinity of the plant that may pose hazards to the facilities and/or operations of the plant have been evaluated.

The distance of 4600 feet has been established as a limit beyond which a possible pipeline rupture followed by an explosion under conservative weather conditions does not govern the design of the plant. Since the pipelines that existed prior to the construction of the plant have been relocated (see Subsection 2.2.2.3), and the closest of these pipelines passes 4650 feet from the site, explosions do not pose any hazard to the plant.

Evergreen FS maintains a large propane tank at their facility in DeWitt. The Evergreen FS facility in Dewitt is located approximately 2.5 miles from Clinton Power Station. The propane stored at Evergreen FS has been considered for hazard from a possible explosion. The propane stored and used by Evergreen FS is transported to the location in DeWitt by truck on Highway 54 and Route 48. At the closest approach of the closest highway to CPS, Highway 54, the risk of an explosion involving approximately 90 tons of hydrocarbon fuel (standard tank trucks do not carry this large a quantity of material) has been reviewed and the safety-related structures at the site are capable of withstanding well in excess of the overpressure. CPS has determined by a bounding calculation (reference 12) that the amount of propane stored at DeWitt is below a level that was calculated to be safe. The amount of propane stored is less than 1,000,000 pounds of propane. The calculated level that was found to be safe was less than 13,240,000 pounds of propane. Therefore there is no risk of damage to the safety-related parts of the plant from a propane explosion at the Evergreen FS facility in DeWitt.

2.2.3.1.2 Flammable Vapor Clouds (Delayed Ignition)

Two potential sources for this type of hazard have been identified: (1) accidents resulting from shipment of compressed flammable gas on the Canadian National/Illinois Central Railroad; and (2) rupture of the pipeline carrying the liquefied petroleum gas.

A conservative estimate for probability of overpressure exceeding 1 psi at CPS due to source (1) is shown to be less than 1 x 10-6 per year. Therefore, the requirement of Regulatory Guide 1.91 is met by the low probability of accidents on the Illinois Central Gulf Railroad and this source is not considered a design basis event. Source (2), namely, the flammable vapor clouds resulting from pipeline rupture, does not pose a hazard to the plant. As discussed in Subsection 2.2.3.1.1, there is a sufficient distance between the pipeline and the site; therefore, source (2) is not considered a design basis event.

2.2.3.1.3 Toxic Chemicals

The potential for the release of toxic chemicals in the vicinity of the plant, and their effect on the habitability and protection of the control room during and after such a release, has been evaluated.

Van Horn - DeWitt is the only facility within five miles of the site which manufactures, uses, or stores toxic chemicals. Van Horn - DeWitt is a distributor of agricultural products and chemicals (such as pesticides, herbicides, and fertilizers) and their facility in DeWitt is located approximately 2.5 miles from Clinton Power Station. CPS reviewed a list of chemicals distributed by Van Horn - DeWitt, and determined that with the exception of anhydrous ammonia, none of the chemicals require evaluation for their potential effect on control room habitability (due to an accidental spill or release) in accordance with Regulatory Guide 1.78.

Calculations (Reference 11) show the postulated accidents of the anhydrous ammonia nurse tanks and tanker trucks used by farmers and suppliers do not adversely affect the control room habitability.

Reference 14 concluded that all the identified toxic chemicals (transported via roadways) do not need further evaluation.

The ICG line running parallel to State Route 54, the Gilman Line, is used to transport numerous commodities including hazardous materials.

Reference 14 concluded that all the hazardous chemical, being transported via rail, do not need further evaluation.

The following analysis (analysis of chemicals transport via rail between year period of December 1, 1981 to November 30, 1982) is historical based on the results from Reference 14.

IPC performed a comprehensive survey of the Gilman Line from ICG shipping records for the one year period of December 1, 1981 to November 30, 1982. The survey found 19 hazardous materials shipped at least thirty times per year (based on evaluation criteria dictated by Reference 4). These 19 chemicals are listed in Table 2.2-7.

Since a significant fraction of the control room air would not be displaced as a result of their release, simple asphyxiants (chemicals that have no specific toxic effects but act by displacing oxygen in the lungs) were eliminated from the chemicals listed in Table 2.2-7. Up to a third of the air in a room can be displaced by a simple asphyxiant before a human being will experience adverse effects (Reference 5).

None of the asphyxiants in Table 2.2-7 will enter the control room in sufficient quantities to displace one-third of the air. On this basis, butane, propylene and butene were eliminated (Reference 5 and Reference 6).

Several chemicals were eliminated from consideration on the basis of toxicity information. Threshold limit values (TLV) indicate the maximum concentration of a chemical to which a human can be safely exposed for several hours daily over long periods of time. Chemicals with low toxicity levels cause slight changes which are readily reversible and disappear after the end of the exposure. The chemicals meeting this qualification were isobutane, propane, and liquid petroleum gas (Note: Liquified butane gas was separately eliminated because of its asphyxiiant character) (Reference 5).

NRC Guidance regulations (Reference 4) state that liquids with vapor pressures less than 10 torr may also be eliminated from further considerations. Sulfuric acid, monoethanolamine and corrosive liquid N.O.S. (either sulfuric acid or sodium hydroxide) all have vapor pressures less than 10 torr at 100ºF. Sodium nitrate was eliminated from consideration because it is a solid at ambient temperatures.

The remaining chemicals were subjected to a diffusion analysis as outlined in Reference 4. A diffusion simulation model was developed to calculate the concentration in the control room of a chemical released a specific distance from the ventilation intake. This model and the calculated results were independently reviewed by Reference 13. The computed control room concentration was then compared to the maximum concentration tolerable by human beings for an acute exposure to determine whether a release of shipment quantity would cause the control room to become uninhabitable.

The following assumptions were used in this analysis:

a. Instantaneous spill of total contents of a tank containing the chemical.

b. Ground release of tank car contents.

c. Control room intake was modeled as being directly downwind of the point of chemical release with no intervening structures.

d. The chemical is a gas at the input temperature and 14.7 psia but was stored or transported as liquid under pressure.

e. Instantaneous release results in a puff or finite volume described by the puff model for atmospheric dilution in Appendix B of Reference 4.

f. The diffusion equation for an instantaneous (puff) ground level release used in the program was taken directly from Appendix B of Reference 4. The "y" and "z" terms in the diffusion equation were assumed to be zero. This assumption centers the puff at the control room intake in the horizontal crosswind and vertical directions.

g. The relationship x = D - ut as defined in Appendix B of Reference 4 was directly substituted for the "x" term in the equation.

h. The value calculated by the equation represents the chemical concentration at the intake to the control room. The program uses the concentration at the intake and the control room ventilation characteristics to determine the chemical concentration inside the control room. Concentration levels are calculated for various equally spaced wind speeds up to the maximum wind speed supplied as inputs into the program.

The use of these assumptions made the analysis very conservative. The concept is based on the basic premise that if an accident occurs, it will occur in the worst possible way and under the worst possible meteorological conditions. Therefore, the effects on the control room habitability will be worse than what would normally be anticipated.

Computer analysis revealed that in the event of a railcar spill, insufficient amounts of several of the remaining toxic chemicals would reach the control room to be hazardous in terms of acute exposure. These chemicals are: propylene oxide, vinyl acetate, carbon tetra chloride, petroleum naphtha (a mixture of hydrocarbons consisting of pentane, hexane and heptane), formaldehyde (a gas, but is shipped as an aqueous solution between 37% and 50%), denatured alcohol and alcohol N.O.S. (ethyl alcohol, anhydrous, denatured in part with petroleum products and/or chemicals, not to exceed 5%). Table 2.2-8 compares the computed concentration of the first 5 chemicals at the control room intake to the maximum allowable concentrations for acute exposures to these chemicals. In all cases shown, the computed concentrations were lower than the acute exposure concentrations. Computed concentrations at the control room intake of the last 2 chemicals, denatured alcohol and alcohol N.O.S., were determined both with respect to the major constituent, pure ethyl alcohol (see Table 2.2-8) and with respect to toxic denaturants mixed with ethyl alcohol in quantities greater than 1% - Benzene, Butyl Alcohol, Chloroform, Ethyl Ether, Formaldehyde, Heptane, Methyl Alcohol, Methyl Isobutyl Ketone, and

Toluene (see Table 2.2-9). In all cases shown, the computer concentrations were lower than the acute exposure concentrations.

The largest shipment of petroleum naphtha was 97 tons. Three separate analyses were made assuming the 97 ton shipment consists entirely of one of the three constituents in each case. Results of these analyses are summarized in Table 2.2-8. Two separate computer analyses were performed for formaldehyde, one as a 98 ton release of 37% formaldehyde and the second as a 50% solution.

The analysis evaluated alcohol N.O.S. under the common category of denatured alcohol's and assumed a release of 100 tons of pure ethyl alcohol. Most denaturants were eliminated from consideration since they are generally in quantities of less than 1%. Of the remaining denaturants generally used in quantities greater than 1%, those considered non-toxic were eliminated. A diffusion simulation run was then performed for the release of only the denaturant with the highest vapor pressure, because the rates of evaporation of the various denaturants will vary with the vapor pressure of the pure denaturant and the weight percent in the denatured alcohol solution. Results of the simulation for the denaturant with the highest vapor pressure, methyl alcohol, in quantities corresponding to the maximum weight percents of each of the nine denaturants, are presented in Table 2.2-9.

Anhydrous ammonia and bromine were the only two chemicals remaining from Table 2.2-7 that were toxic and were evaluated by the computer analysis. The analyses revealed that either chemicals would render the control room uninhabitable if a railcar containing the maximum shipment quantity were to spill its entire contents. These two chemicals required further evaluation to determine if a significant probability existed for an unacceptable transportation accident.

The probabilistic risk assessment analysis employs two conservative and cross-checking methods to calculate the probability of a railcar rupture and toxic materials release serious enough to affect the habitability of the CPS Control Room.

The first probability calculation was a function of the probability of release per car mile and the shipping frequency in cars per year. This probability is equal to:

$$Pa = Pr(C) \times F(C) \times \sum_{D=1}^{8} L(D) \times Pw(D) \qquad \text{(Equation 1)}$$

where:

$$Pa = \text{probability of accident} \qquad \frac{[\text{releases}]}{[\text{year}]}$$

$$Pr(C) = \text{probability of release} \qquad \frac{[\text{releases}]}{[\text{car mile}]}$$

$$F(C) \;=\; \text{frequency of shipment} \qquad \frac{[\text{cars}]}{[\text{year}]}$$

L(D) = length of track under consideration (function of wind direction)

Pw (D) = probability that a wind of any stability class and any velocity class is blowing in a direction such that a toxic chemical release is carried toward the control room air intake (function of wind direction) dimensionless

D is the direction from which the wind is blowing (W, WNW, etc.). Only those eight wind directions from which a wind could blow from the railroad towards the plant were included.

The second probability calculation was a function of the probability of release per ton mile and the shipping frequency in tons per year. This probability was computed by:

$$Pa = Pr\,(T) \;\times\; F\,(T) \;\times\; \sum_{D=1}^{8} \; L\,(D) \;\times\; Pw\,(D) \qquad \text{(Equation 2)}$$

where:

Pa, L (D), Pw (D) and D are as defined before and,

$$Pr\,(T) \;=\; \text{probability of release} \qquad \frac{[\text{release}]}{[\text{ton mile}]}$$

$$F\,(T) \;=\; \text{frequency of shipment} \qquad \frac{[\text{tons}]}{[\text{year}]}$$

Anhydrous ammonia, classified as a non-flammable gas, has accident release frequencies of (Reference 10):

$$Pr\,(C) \;= 0.019 \times 10^{-6} \qquad \frac{[\text{releases}]}{[\text{car mile}]}$$

$$Pr\,(T) \;= 0.27 \times 10^{-9} \qquad \frac{[\text{releases}]}{[\text{ton mile}]}$$

From Table 2.2-7, anhydrous ammonia has shipping frequencies of:

$$F\,(C) \;= 37 \qquad \frac{[\text{cars}]}{[\text{year}]}$$

$$F\,(T) \;= 3{,}119 \qquad \frac{[\text{tons}]}{[\text{year}]}$$

For the Clinton Power Station Site:

$$\sum_{D=1}^{8} L\,(D) \times Pw\,(D) = 0.5769 \text{ miles}$$

Substituting into equations 1 and 2 the probability of an anhydrous ammonia release:

$$Pa_{(1)} = 0.019 \times 10^{-6} \left[\frac{\text{releases}}{\text{car mile}}\right] \times 37 \left[\frac{\text{cars}}{\text{year}}\right] \times 0.5769 \quad \text{miles}$$

$$= 4.06 \times 10^{-7} \left[\frac{\text{releases}}{\text{year}}\right]$$

$$Pa_{(2)} = 0.27 \times 10^{-9} \left[\frac{\text{releases}}{\text{ton mile}}\right] \times 3119 \left[\frac{\text{tons}}{\text{year}}\right] \times 0.5769 \quad \text{miles}$$

$$= 4.86 \times 10^{-7} \left[\frac{\text{releases}}{\text{year}}\right]$$

Bromine, classified as a corrosive, has accident release frequencies of (Reference 10):

$$Pr\,(C) = 0.090 \times 10^{-6} \left[\frac{\text{releases}}{\text{car mile}}\right]$$

$$PR\,(T) = 1.10 \times 10^{-9} \left[\frac{\text{releases}}{\text{ton mile}}\right]$$

From Table 2.2-7, bromine has shipping frequencies of:

$$F\,(C) = 34 \left[\frac{\text{cars}}{\text{year}}\right]$$

$$F\,(T) = 1{,}340 \left[\frac{\text{tons}}{\text{year}}\right]$$

Substituting into the equations, the probability of a bromine release:

$$Pa_{(1)} = 0.090 \times 10^{-6} \left[\frac{\text{releases}}{\text{car mile}}\right] \times 34 \left[\frac{\text{cars}}{\text{year}}\right] \times 0.5769 \quad \text{miles}$$

$$= 1.77 \times 10^{-6} \left[\frac{\text{releases}}{\text{year}}\right]$$

$$Pa_{(2)} = 1.10 \times 10^{-9} \left[\frac{\text{releases}}{\text{ton mile}}\right] \times 1{,}340 \left[\frac{\text{tons}}{\text{year}}\right] \times 0.5769 \quad \text{miles}$$

$$= 8.50 \times 10^{-7} \left[\frac{\text{releases}}{\text{year}}\right]$$

The computed release probabilities for anhydrous ammonia and bromine demonstrate that the expected rates of event occurrences, leading to potential consequences in excess of 10 CFR Part 100 exposure guidelines, are approximately 10^{-6} per year or less. These frequencies are acceptable if, when combined with reasonable qualitative arguments, the realistic probabilities can be shown to be lower. The realistic probabilities can be considered lower because of the following conservatisms:

a. No credit was taken in the release probabilities for improved safety in tank car modifications. The release probability data are based on the period 1971-1977 prior to completion of tank car safety modifications.

b. No credit was taken for unstable atmospheric conditions which would not be conducive to a slow diffusion of the toxic chemicals.

c. No credit was taken for the effects of the lake which could enhance diffusion of the plume in the vertical direction, and

d. No credit was taken for operation incapacitation events that would not result in exposures in excess of guidelines. The analysis assumed all events resulted in an overexposure.

Since the probability analysis used a conservative approach with conservative data and resulted in calculated probabilities of anhydrous ammonia and bromine release, of 10 per year or less, these compounds are exempt from consideration as design basis accidents.

In summary, the releases of hazardous materials shipped by rail in the vicinity of CPS need not be considered as design basis accidents.

2.2.3.1.4 Fires

No external fire hazard can threaten plant safety, since no chemical plants or oil storage are located near the plant. Forest or brush fires cannot pose any danger because of the site landscaping.

2.2.3.1.5 Collisions with Intake Structure

There is no potential for a ship or a barge to impact on the intake structure since only small recreational boats are operated near the site.

2.2.3.1.6 Liquid Spills

No potential for an accidental release of oil or other liquids 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.

2.2.3.2 Effects of Design Basis Events

All hazardous materials stored or shipped in the vicinity of CPS were evaluated in Subsection 2.2.3.1.3 for their toxic potential on control room habitability. Based on this evaluation, releases of hazardous materials in the vicinity of CPS need not be considered as design basis accidents.

2.2.3.3 Reference

1. Illinois Airport Directory 2000-2001, Illinois Department of Transportation.

2. Schowalter, Staff member of Decatur Airport, telephone conversation of April 19, 1979, with S. A. Hallaron, Sargent & Lundy, Cultural Resource Analyst.

3. Staff Secretary, Staff member of Bloomington-Normal Airport, telephone conversation of April 19, 1979 with S. A. Hallaron, Sargent & Lundy, Cultural Resource Analyst.

4. Regulatory Guide 1.78, "Assumptions for Evaluating the Habitability of a Nuclear Power Plant Control Room During a Postulated Hazardous Chemical Release;" June 1974.

5. Sax, N. Irving, Dangerous Properties of Industrial Materials, Third Edition, Reinhold Book Corp., New York, N.Y., 1968.

6. Broker, William and A. L. Mossman, Matheson Gas DataBook, Fifth Edition, September 1971.

7. Patty's Industrial Hygiene and Toxicology, Volume 2A, Third Revised Edition, G. D. Clayton and F. E. Clayton, Editors, John Wiley and Sons, New York, N.Y., 1981.

8. Browning, Ethel, Toxicity and Metabolism of Industrial Solvent, Elsevier Publishing Company, New York, N.Y. (Also Amsterdam and London), 1965.

9. Registry of Toxic Effects of Chemical Substances, U.S. Department of H.E.W., Public Health Service, Center for Disease Control prepared for Table 2.2-2 National Institute for Occupational Safety and Health, 1980.

10. Nayak, P. R. and D. W. Palmer, Issues and Dimensions of Freight Car Size: a compendium, U.S. Department of Transportation, Federal Railroad Administration Report No. FRA/ORD--79/56, October, 1980.

11. IP Nuclear Station Engineering Department calculation MAD 91-073 revision 3; "Control Room Habitability Study - Anhydrous Ammonia," December, 1994.

12. IP Nuclear Station Engineering Department calculation IP-M-0341; "Propane Storage at Evergreen F.S., DeWitt, IL," November, 1994.

13. IP Nuclear Station Engineering Department Calculation MAD 81-611 "Hazardous Chemical Analysis/Independent Review".

14. 2013 Clinton Power Station Hazardous Chemical Survey, VC-94, Rev 0.

TABLE 2.2-1
MILITARY ARMORIES WITHIN 50 MILES

LOCATION	NUMBER OF ARMORIES	DISTANCE RELATIVE TO THE SITE
Bloomington	2	23 miles NNW
Decatur	2	25 miles SSW
Springfield	3	49 miles WSW
Champaign	1	30 miles E

Table 2.2-2
INDUSTRIES WITHIN 5 MILES AND INDUSTRIES WHICH MAY IMPACT CLINTON POWER STATION

INDUSTRY	NUMBER OF EMPLOYEES (APPROX.)	PRODUCT(S)
Evergreen FS (DeWitt)	N/A	Propane storage
Evergreen FS (Wapella)	20	Agricultural chemicals and fertilizers
Maroa Ag Fertilizer Services	5	Agricultural chemicals and fertilizers
Van Horn - DeWitt	3	Agricultural chemicals and fertilizers
Weldon Fertilizer and Lumber Co.	10	Agricultural chemicals and fertilizers; Lumber Products
ADM Grain	N/A	Propane Storage
Tate and Lyle Parnell Grain	N/A	Propane Storage

Source: DeWitt County Hazardous Material Contingency Plan, Annex V, Appendix 3, May 1995.

TABLE 2.2-3 has been deleted.

TABLE 2.2-4
PIPELINES WITHIN 5 MILES

PIPELINE COMPANY	PIPE DIAMETER (inches)	MATERIAL CARRIED	YEAR PIPE INSTALLED	OPERATING PRESSURE (psi)	DEPTH OF BURIAL	LOCATION AND TYPE OF ISOLATION VALVE	APPROXIMATE DISTANCE FROM PLANT (feet)
Buckeye Partners Pipeline Company	14	Refined petroleum products	1976	1000-1100	> 36 in.	Manual control both sides of Lake Clinton	4,650
Explorer Pipeline Company	24	Refined petroleum products	1976	750-900	> 36 in.	Manual control both sides of Lake Clinton	13,7000
Conoco-Phillips Pipeline Company (2 lines)	8	Refined petroleum products	1976	750-1100	> 36 in.	Manual control both sides of Lake Clinton	13,700
Ameren Illinois	2	Natural gas	1966	450	36 in.	None	12,000

TABLE 2.2-5
POTENTIALLY HAZARDOUS CHEMICALS
STORED WITHIN THE PROTECTED AREA

CHEMICAL	NOMINAL QUANTITY	LOCATION
Caustic (50% & 25% Solution)	10,000 gal 100 gal	Radwaste Building Elevation 702 ft Makeup Water Pump House 737 ft
Sulfuric Acid	10,000 gal	Radwaste Building Elevation 702 ft and various other battery rooms.
Fuel Oil	148,350 gal	Diesel-Generator Building/Screen House
Lubrication Oil	30,000 gal 12,000 gal	Radwaste Building Elevation 737 ft Turbine Building Elevation 762 ft
Corrosion Inhibitor	4,400 gal	Raw Water Treatment Bldg.
Acetylene	3,000 ft^3	Radwaste Building Elevation 737 ft
Scale Inhibitor	3,150 gal	Raw Water Treatment Bldg.
Sodium Bisulfite	2,500 gal 622 lb	South of Control Building Makeup Water Pump House
Sodium Hypochlorite	8,700 gal 55 gal	Raw Water Treatment Bldg. Makeup Water Pump House 737 ft
Dispersant	4,400 gal	Raw Water Treatment Bldg.
Aluminum Chloride Hydroxide	600 lb	Makeup Water Pump House
Sodium Hydoxide	564 lb	Makeup Water Pump House

Note: This table is here as historical information.

TABLE 2.2-6
CHEMICALS STORED WITHIN THE PROTECTED AREA

CHEMICAL	NOMINAL QUANTITY
Caustic (50% & 25% Solution)	10,000 gal
Sulfuric Acid	10,500 gal
Polyacrylamide	165 gal
Trisodium Phosphate	1,000 lb
Sodium Nitrite	500 lb
Fuel Oil	148,350 gal
Lubrication Oil	42,000 gal
Glycol	1,000 gal
Carbon Dioxide	34,000 lb (3 tanks)
Acetylene	3,000 ft^3 (20 tanks)
Oxygen	7,000 ft^3 (23 tanks)
Nitrogen	11,300 ft^3 (50 tanks)
Argon	9,000 ft^3 (30 tanks)
Halon 1301	2,200 lb
Scale Inhibitor	3,150 gal
Sodium Bisulfite	3,122 gal
Sodium Hypochlorite	8,755 gal
Polymer/Coagulant	500 gal
Corrosion Inhibitor	4,400 gal
Corrosion Inhibitor (Non-hazardous)	8,700 gal
Cosmetic Ingredient	4,400 gal
Deposit Control Agent	3,150 gal
Water Treatment	450 lb
Sodium Hydoxide	564 lb
Azoles	3,150 gal
Phosphoric Acid	525 lb
Aluminum Chloride Hydoxide	600 lb

Note: This table is here as historical information.

TABLE 2.2-7
HAZARDOUS MATERIALS SHIPMENTS

DESCRIPTION OF COMMODITY	CARLOADS	TONS
Butane	443	31,146
Propylene	801	57,132
Liquefied Petroleum Gas (butene gas, liquefied)	345	24,459
Isobutane	793	57,001
Propane	164	11,559
Liquefied Petroleum Gas	885	61,816
Sulfuric Acid	156	13,831
Monoethanolamine	44	3,391
Corrosive Liquid, N.O.S.	34	2,621
Sodium Nitrate	34	1,980
Propylene Oxide	77	5,164
Vinyl Acetate	137	10,769
Carbon Tetrachloride	185	15,560
Petroleum Naphtha	47	3,468
Formaldehyde (or) formalin solution (in containers over 100 gallons)	38	3,227
Denatured Alcohol	56	3,874
Alcohol, N.O.S. (ethyl alcohol, anhydrous, denatured in part with petroleum products and/or chemicals not to exceed five percent)	60	4,817
Anhydrous Ammonia	37	3,119
Bromine	34	1,340

Note: Hazardous Materials Shipments with a Frequency of 30 or More Cars Per Year Over the Illinois Central Gulf-Gilman Line from December 1, 1981 to November 30, 1982. Per Reference 14, all the hazardous chemical shipments via rail were less than 30 times per year. The information provided in this table is retained for historical purposes.

TABLE 2.2-8
CALCULATIONS OF TOXIC CHEMICAL CONCENTRATIONS AT CLINTON STATION

CHEMICAL	AMOUNT OF * CHEMICAL EVALUATED	CONCENTRATION AT CONTROL ROOM INTAKE (CALCULATED)	MAXIMUM ALLOWABLE CONCENTRATION FOR ACUTE EXPOSURES	REFERENCE/COMMENTS
Propylene Oxide	121 tons	8.43×10^{-5} lb/ft^3 (562 ppm)	2.25×10^{-4} lb/ft^3 (1500 ppm)	Reference 7
Vinyl Acetate	101 tons	1.71×10^{-5} lb/ft^3 (77 ppm)	No acute exposure limits were found	Reference 8 (reports it to be a "relatively non-toxic material.")
Carbon Tetrachloride	130 tons	4.42×10^{-5} lb/ft^3 (109 ppm)	6.09×10^{-4} lb/ft^3 (1500 ppm)	Reference 5
Pentane (Petroleum Naphtha)	97 tons	1.707×10^{-4} lb/ft^3 (927 ppm)	2.21×10^{-4} lb/ft^3 (1200 ppm) = 2 X TLV**	Reference 9
Hexane (Petroleum Naphtha)	97 tons	5.097×10^{-5} lb/ft^3 (232 ppm)	1.10×10^{-4} lb/ft^3 (500 ppm) = TWA**	Reference 9
Heptane (Petroleum Naphtha)	97 tons	2.371×10^{-5} lb/ft^3 (95 ppm)	2.00×10^{-4} lb/ft^3 (800 ppm) = 2 X TLV**	Reference 9
37% Formaldehyde (Formalin)	98 tons	4.496×10^{-8} lb/ft^3 (0.6 ppm)	7.49×10^{-7} lb/ft^3 (10 ppm)	Reference 4
50% Formaldehyde	98 tons	5.941×10^{-8} lb/ft^3 (0.8 ppm)	7.49×10^{-7} lb/ft^3 (10 ppm)	Reference 4
Ethyl Alcohol	100 tons	4.97×10^{-6} lb/ft^3	5.87×10^{-4} lb/ft^3 (5000 ppm)	Reference 4

* Maximum Shipping Weight from Survey.

** If an acute exposure limit could not be found, a value of 2 X TLV (Threshold Limit Value for an 8-hour, daily exposure) or the TWA (Time Weighted Average for lengthy exposure) was used. These values are very conservative.

Note: This table is here as historical information.

TABLE 2.2-9
CALCULATIONS OF TOXIC CHEMICAL
CONCENTRATION AT CLINTON STATION

CHEMICAL	AMOUNT OF CHEMICAL EVALUATED	CONCENTRATION AT CONTROL ROOM INTAKE (CALCULATED)	MAXIMUM ALLOWABLE CONCENTRATION FOR ACUTE EXPOSURES
Methyl Alcohol (as a worst case for denatured alcohol)	100 tons	0.2234×10^{-4} lb/ft^3 (275 ppm)	2 x TLV* 0.3015×10^{-4} lb/ft^3 (400 ppm)

DENATURED ALCOHOL: Concentrations at the control room intake for the following denaturants were estimated by scaling down the concentration for a 100-ton methyl alcohol spill to the maximum amount of each denaturant found in 100 tons of denatured ethyl alcohol.

DENATURANT	MAXIMUM % BY WEIGHT IN ETHYL ALCOHOL FOUND IN LITERATURE	CONCENTRATION AT CONTROL ROOM INTAKE FOR AN EQUIVALENT AMOUNT OF METHANOL	MAXIMUM ALLOWABLE CONCENTRATION FOR ACUTE EXPOSURES
Benzene	5.27%	13 ppm	2 x TLV* = 50 ppm
Butyl Alcohol	2.79%	7 ppm	2 x TLV* = 200 ppm
Chloroform	8.5%	12 ppm	2000 ppm
Ethyl Ether	8.15%	25 ppm	800 ppm
Formaldehyde	4.37%	8 ppm	10 ppm
Heptane	5%	16 ppm	2 x TLV* = 1000 ppm
Methyl Isobutyl Ketone	5%	14 ppm	2 x TLV* = 200 ppm
Toluene	5.07%	13 ppm	2 x TLV* = 400 ppm

* If an acute exposure limit could not be found, a value of 2 x TLV (Threshold Limit Value for an 8-hour, daily exposure) was used. This value is very conservative.

NOTE: All denaturant maximum allowable concentrations were taken from Reference 5, except for ethyl ether and formaldehyde, which were taken from Reference 4.

This table is here as historical information.

2.3 METEOROLOGY

2.3.1 Regional Climatology

This section provides a description of the general climate of the Clinton Power Station (CPS) site region, as well as the regional meteorological conditions used for design and operating-basis considerations. A climatological summary of normal and extreme values of several meteorological parameters is presented for the first-order National Weather Service Stations at Peoria, Illinois and Springfield, Illinois. Further information regarding the regional climatology was derived from pertinent documents which are referenced in the text.

2.3.1.1 General Climate

The CPS site is located near the geographical center of Illinois, approximately 55 miles southeast of the first-order National Weather Service Station at Peoria, Illinois, and 49 miles east-northeast of the first-order National Weather Service Station at Springfield, Illinois. General climatological data for the region were obtained from U.S. Environmental Science Services Administration (ESSA) Climate of Illinois report (Reference 1), and from the Local Climatological Data Annual summaries for the first-order weather stations at Peoria (Reference 2), and Springfield (Reference 3). The climatic data from Peoria and Springfield are considered to be representative of the climate at the CPS site.

The climate of central 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 central Illinois climate is due to its location in a confluence zone (particularly during the cooler months) between different air masses (Reference 4). The specific air masses which affect central 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 Mexico and eastern North Pacific Ocean; and continental polar and continental arctic air which originates 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 central 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 4).

The major factors controlling the frequency and variation of weather types in central 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 is 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 United States-Canadian border. The average frequency of passage of storm systems along this zone is about once every 5 to 8 days. The storm systems are most frequent during the 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, or snow) and periods of clear, crisp, and cold weather. Springtime precipitation is primarily showery in nature. The frequent 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 and 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 this season which ends rather abruptly with the returning storminess that 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 interspersed 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 central 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 both Peoria and Springfield. 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 south-southwest at Springfield. The monthly average wind speed is highest during late winter and early spring at both stations, with the prevailing directions from the west-northwest and the south at Peoria, and the northwest and south at Springfield.

Table 2.3-1 presents a summary of climatological data from meteorological stations surrounding the CPS site. The annual average temperature is 50.8° F at Peoria and 52.7° F at Springfield. Monthly average temperatures in the CPS site region range from the middle twenties in January to the middle seventies in July. Extreme temperatures in the region range from a maximum of 103° F (Peoria) and 112° F (Springfield) to a minimum of -20° F (Peoria) and -22° F (Springfield). Maximum temperatures in the CPS site region equal or exceed 90° F with an average of from 17 to 28 times per year. Minimum temperatures in this region are less than or equal to 32° F for an average of from 119 to 132 times per year (References 2 and 3).

Humidity varies with wind direction, being lower with west or northwest winds and higher with east or south winds. The early morning relative humidity is highest during the late summer, with an average of 87% at both Peoria and Springfield. The relative humidity is highest throughout the day during December, ranging from 83% in early morning to 72% at noon at both Peoria and Springfield. Heavy fog with visibility less than 1/4 mile is rare, having an average occurrence of 21 times per year at Peoria and 18 times per year at Springfield. It occurs most frequently during the winter months (References 2 and 3).

Annual precipitation at the CPS site area averages about 35 inches per year. For the 40-year period (1937-1976) the minimum annual precipitation was 23.99 inches at Peoria (1940), and 22.88 inches at Springfield (1940). For the same period, the maximum annual precipitation was 50.22 inches at Peoria (1973), and 44.72 inches at Springfield (1941). On the average, about 45% of the annual precipitation occurs in the 4 months of April through August in the CPS site region. However, no month in this region averages less than 4% of the annual total. Monthly precipitation totals have ranged from 13.09 inches (Peoria) to 0.03 inches (Peoria). The maximum 24-hour precipitation at either station was 5.52 inches, recorded at Peoria in May 1927. Snowfall commonly occurs from November through March, with an annual average of 23.4 inches at Peoria, and 22.3 inches at Springfield. The monthly maximum snowfall of 18.9

inches at Peoria, and 22.7 inches at Springfield, occurred in December 1973. The 24-hour maximum snowfall, which also occurred in December 1973, was 10.2 inches at Peoria, and 10.9 inches at Springfield (Reference 5).

The terrain in central 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 50 days per year at Springfield (1948-1976)

(References 2 and 3). Approximately 41% of the annual precipitation in the Clinton area falls during thunderstorms (Reference 6). Thunderstorms occur most frequently during the months of June and July; 9 and 8 days per month respectively at Peoria, and 8 and 9 days per month respectively at Springfield. Peoria and Springfield average 5 or more thunderstorm days per month throughout the season from April through September. Both stations average one or less thunderstorm days per month from November through February (References 2 and 3). 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 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 by longitude). The report shows that during this 13-year period the 1° square containing the CPS site had 15 hailstorms producing hail 3/4-inch in diameter or greater, 26 occurrences of winds of 50 knots or greater, and 42 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 the CPS site region, an average of about 22 hail days per 10-year period occurs, with about 55% of all hail days occurring in the spring (Reference 9). Total hailstorm life at a point averages about 7 minutes, with maximum storm life reported as generally 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:

$$N = (0.1 + 0.35 \sin \varnothing) \quad (0.40 \pm 0.20)$$

where:

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

and

$\varnothing$= geographical latitude.

For the Clinton Power Station, which is located at 40° 10'19.5" north latitude, the frequency of lightning flashes (N) ranges from 0.065 to 0.195 flashes per thunderstorm day per km^2. The value 0.195 is used as the most conservative estimate of lightning frequency in the calculations that follow.

Taking the annual average number of thunderstorm days in the site region as 50 (at Springfield), the mean frequency of lightning flashes per km^2 per year is 9.8 as calculated below:

$$\frac{0.195\,\text{flashes}}{\text{thunderstorm day} \bullet \text{km}^2} \cdot \frac{50\,\text{thunderstorm days}}{\text{year}}$$

$$= \frac{9.8\,\text{flashes}}{\text{km}^2 \bullet \text{year}}$$

The area of the CPS site is approximately 14,000 acres, or about 56.7 km^2. Hence the expected frequency of lightning flashes at the site per year is 555, as calculated below:

$$\frac{9.8\,\text{flashes}}{\text{km}^2 \bullet \text{year}} \cdot 56.7\,\text{km}^2 \quad = 555\,\frac{\text{flashes}}{\text{year}}$$

The exclusion area at the CPS site has a radius of 975 meters (3199 feet), or an area of about 3.0 km^2. Hence the expected frequency of lightning flashes in the exclusion area per year is 29, as calculated below:

$$\frac{9.8\,\text{flashes}}{\text{km}^2 \bullet \text{year}} \cdot 3.0\,\text{km}^2 \quad = 29\,\frac{\text{flashes}}{\text{year}}$$

2.3.1.2.2 <u>Tornadoes and Severe Winds</u>

Illinois ranks eighth in the United States in average annual number of tornadoes. An average of ten tornadoes per year occur on 5 days, based on the period-of-record 1916-1969 (Reference 11). The majority of Illinois tornadoes (65%) occur during the months of March through June. The statewide probability of a tornado occurrence is greatest during the 7-day period of April 15-21. 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).

Figure 2.3-1 presents the total number of tornadoes (1916-1969) for each county in Illinois. There were 36 tornadoes in the period which originated in the five-county areas (DeWitt, McLean, Logan, Macon, and Piatt) surrounding and including the CPS site. Three tornadoes originated in De Witt County during the 54-year period.

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 by longitude) for the period 1953-1962. For the 1° square (3634 mi^2 in area) containing the CPS site, Thom computed an annual average of 1.9 tornadoes. Assuming 2.82 mi^2 is the average tornado path area (Reference 12), the mean probability of a tornado occurring at any point within the 1° square containing the CPS site in any given year is calculated to be .0015. This converts to a mean recurrence interval of 680 years. Using the same annual frequency but an average area of tornado coverage of 3.5 mi^2 (from Wilson and Changnon, Reference 11), the mean probability of a tornado occurrence is .0018.

More recent data (Reference 8) containing tornado frequencies for the period 1955-1967 indicate an annual tornado frequency of 3.2 for the 1° square containing the CPS site. This frequency, with Wilson and Changnon's average path area of 3.5 mi^2, results in an estimated mean tornado probability of .0031, with a corresponding mean return period of 325 years.

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 CPS 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. A conservatively high estimate can be taken to be .0031, with a corresponding mean return period of 325 years.

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

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

e. rate of pressure drop = 2.0 psi/sec

A design wind velocity of 85 mph (100-year recurrence interval) was used in the design of Clinton Power Station Seismic Category I structures. This design wind velocity is estimated from the analysis presented in Figure 2 of the "American National Standard Building Code Requirements for Minimum Design Loads in Buildings" (Reference 14). The vertical velocity distribution and gust factors employed for the design wind loading are those specified in 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 mi2, with more than 6 inches of snow covering 7500 mi^2. Central Illinois (including the CPS site) had 107 occurrences of a 6-inch snow or glaze damage area during the years 1900-1960. About 42 of those storms deposited more than 6 inches of snowfall in De Witt County (Reference 15).

In the Springfield area, the maximum 24-hour snowfall was 15.0 inches, and the maximum monthly snowfall was 24.4 inches, both of which occurred in February 1900. On the average, heavy snows of 4 to 6 inches have occurred one to two times per year (Reference 15).

The 2-day and 7-day maximum snowfall values for selected recurrence intervals in the Clinton Power Station area are as follows (Reference 15):

	2-yr	5-yr	10-yr	20-yr	30-yr	50-yr
2-day	7.0	8.6	10.2	12.1	13.4	15.2
7-day	7.6	10.1	12.8	16.3	18.7	22.0

The listed value is the number of inches of snowfall which would be equalled or exceeded in the given interval of years.

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 32° F from an overlying warm layer of temperature above 32° F. 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, severe glaze storms occur on an average of about three times every 2 years. Statewide statistics indicate that 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 six (1951); in 2 years, nine (1950-1951); in 3 years, ten (1950-1952); and in 5 years, fifteen (1948-1952). 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 central third of Illinois, eleven localized areas received damaging glaze in an average 10-year period; the CPS site area averages slightly over 5 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). Table 2.3-3 presents specific glaze thickness data for the five fastest 5-minute speeds and the speeds with the five greatest measured glazed thickness for 148 glaze storms throughout the country during the period 1926-1937 (Reference 15). Although these data were collected from various locations throughout the United States, they are considered applicable designed 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 100-year return period snowpack of 22 psf (or 22 inches of snowpack) was obtained from the American National Standards building code requirements (Reference 14); however, for design a 100-year return period snowpack of 25 psf is used.

The weight of the accumulation of the winter PMP from a single storm is 79 psf (15.2 inches of precipitable water, or about 152 inches of fresh snow), which was taken as the 48 hour PMP during the winter months (December through March) (Reference 18). Thus the weight of snow and ice on the roof of each safety-related structure can be conservatively estimated as 104 psf.

2.3.1.2.4 Ultimate Heat Sink Design

The meteorological conditions used in evaluation of the performance of the ultimate heat sink were obtained from Peoria, Illinois meterological data for the period of record 1949 through 1971, which was supplemented with meterological data from Springfield, Illinois for the period January 1, 1952 through December 31, 1956. The critical period which showed the maximum station intake temperature was July 1, 1964 through September 30, 1964. The mean values of wind speed, dry bulb temperature, wet bulb temperature, and dew point temperature for this 92-day period were 8.4 mph, 71° F, 64° F, and 59° F, respectively. This period was also found to be a period of high evaporative losses. For further details, see Subsection 9.2.5.

2.3.1.2.5 Inversions and High Air Pollution Potential

Weather records from many U.S. weather stations have been analyzed by Hosler (Reference 19) and Holzworth (Reference 20) with the objective of characterizing atmospheric dispersion potential. The seasonal frequencies of inversions based below 500 feet for the CPS site are shown by Hosler (Reference 19) as:

SEASON	INVERSIONS BELOW 500 FEET % OF TOTAL HOURS	% OF 24-HOUR PERIODS WITH AT LEAST 1 HOUR OF INVERSION
Winter	29	53
Spring	29	67
Summer	33	81
Fall	39	8

Since central Illinois has a primarily continental climate, inversion frequencies are closely related to the diurnal cycle. The less frequent occurrence of storms in summer and early fall 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 give an indication of the vertical depth of atmosphere available for mixing and dispersion of effluents. For the CPS region, the seasonal values of the mean daily mixing depths are (Reference 20):

	MEAN DAILY MIXING DEPTHS (meters)	
SEASON	MORNING	AFTERNOON
Winter	400	690
Spring	490	1500
Summer	330	1600
Fall	390	1200

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

Holzworth has also presented statistics on the frequency of episodes of high air pollution potential, defined as a combination of low mixing depth and light winds (Reference 20). Holzworth's data indicate that during the 5-year period 1960-1964, the region including the CPS site experienced no episodes of 2 days or longer with mixing depths less than 500 meters and winds less than 2 meters per second. There were two such episodes with winds remaining less than 4 meters per second. For mixing heights less than 1000 meters and winds less than 4 meters per second, 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 central Illinois is in a relatively favorable dispersion regime with respect to low frequency of extended periods of high air pollution potential.

2.3.2 Local Meteorology

The onsite meteorological monitoring program began at the Clinton Power Station (CPS) site on April 13, 1972. Onsite meteorological instrumentation is described in Subsection 2.3.3. Data from this installation have been used in preparation of the local meteorological summaries. These data are from a 5-year period of record (April 13, 1972 through April 30, 1977) and, therefore, can be considered representative of long-term site meteorology.

2.3.2.1 Normal and Extreme Values of Meteorological Parameters

2.3.2.1.1 Wind Summaries

Detailed wind records, suitable for the preparation of wind roses, are available from the plant site for April 1972 through April 1977. Monthly and period of record wind roses were constructed for the 33-foot (10 meter) level of the onsite meteorological tower. The period of record wind rose is presented in Figure 2.3-2. The composite monthly wind roses are found in Figures 2.3-3 through 2.3-14.

Seasonal variations are evident from monthly data. Winds from the sector SSE through WNW tend to dominate in most months. Winter months show generally higher wind speeds, fewer calms and more WNW winds than do the summer months.

For the period of record, the following frequencies of occurrence were observed at the CPS site for the specified wind speed intervals:

Wind Speed	Percent of Occurence
< 0.3 mps (calm)	0.3%
0.3 to 1.4 mps	7.7%
1.5 to 3.0 mps	28.2%
3.1 to 5.0 mps	30.7%
5.1 to 8.0 mps	23.7%
> 8.0 mps	9.4%

There were two occurrences of persistence of wind direction for 33 hours (the longest persistence observed). These occurred in two sectors, the SSW and the NE.

2.3.2.1.2 Temperatures

Temperatures at the Clinton Power Station meteorological monitoring site were measured at the 10 meter level of the tower. The average daily temperature for the period of record is 10.5° C (50.9° F). The absolute maximum is 35.2° C (95.4° F) and the absolute minimum is -28.8° C (-19.8° F). Period of record and composite monthly summaries of onsite temperature data are presented in Tables 2.3-4 through 2.3-6.

2.3.2.1.3 Atmospheric Moisture

2.3.2.1.3.1 Relative Humidity

The relative humidity for a given moisture content of the air is inversely proportional to the temperature cycle. A maximum relative humidity usually occurs during the early morning hours, and a minimum is typically observed in the midafternoon. For the annual cycle, the lowest humidities occur in midspring, the winter months experience the highest. Table 2.3-7 presents a relative humidity summary for the Clinton Power Station.

2.3.2.1.3.2 Wet Bulb

The wet bulb temperature is not as strong a function of the ambient temperature as the relative humidity and is used for evaporative cooling system modeling studies. The wet bulb temperature is defined to be the temperature to which an air parcel may be cooled by evaporating water into it at constant pressure until it is saturated. All latent heat utilized in the process is supplied by the air parcel. Summaries of wet bulb temperatures are presented in Table 2.3-8. These values are calculated from the dew point and ambient temperatures, assuming a constant standard sea level pressure of 1013.25 millibars.

2.3.2.1.3.3 Dew Point Temperature

Dew point temperature is a measure of absolute humidity in the air. It is the temperature at which the air must be cooled to cause condensation to occur, assuming pressure and water vapor content remain constant. Composite monthly and period of record dewpoint summaries are presented in Tables 2.3-9 through 2.3-11.

2.3.2.1.3.4 Precipitation

The average yearly precipitation for the period of record for the Clinton Power Station site is 25.47 inches. Period of record and composite monthly precipitation data appear in Table 2.3-12. The months of March and June are the wettest, and December, January, and February are the driest.

2.3.2.1.3.5 Fog

Fog is an aggregate of minute water droplets suspended in the atmosphere near the surface of the earth. According to international definition, fog reduces visibility to less than 0.62 mile. According to United States observing practice, ground fog is a fog that hides less than 0.6 of the sky, and does not extend to the base of any clouds that may lie above it. Ice fog is fog 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 21).

Since local data are not available to assess the fog statistics at Clinton, data are presented for Springfield, Illinois and Peoria, Illinois. Fog is a very local phenomenon; thus, these data should be considered only as regional estimates. The average number of days during which heavy fog (visibility less than 1/4 mile) occurs is as follows (Reference 22).

	SPRINGFIELD	Peoria
January	2	3
February	3	3
March	2	2
April	1	1
May	1	1
June	1/2	1

	SPRINGFIELD	Peoria
July	1	1
August	1	1
September	1	1
October	1	1
November	2	2
December	3	3
Year	18.5	20

The yearly average is 18.5 days at Springfield and 20 days at Peoria; the highest occurrence of fog at both locations is in the winter months.

Tables 2.3-13 and 2.3-14 summarize the occurrence of all fog for Peoria and Springfield, respectively. These summaries were prepared by processing the digital data tapes for these stations. Fog extracted from these tapes were any of the three fogs coded "fog," "ground fog," and "ice fog" which occurred in column 132, "obstruction to vision," on the Airways Surface Observations tapes.

The percentage of the total observations that fog was reported for Peoria and Springfield is given in the first column of Tables 2.3-13 and 2.3-14. The hour and the percentage of observations for that hour of the maximum and minimum fog occurrence are given in the next four columns.

Peoria shows a higher frequency of fog in all months than Springfield. The long-term annual average percent of hourly observations with any intensity of fog for Peoria and Springfield are 11.3% and 9.1%, respectively. The occurrence of prolonged periods of fog is also greater for Peoria. Although information on fog is generally a very local phenomenon, the expected occurrences at the Clinton Power Station should be within the range represented by these two stations.

2.3.2.1.4 Atmospheric Stability

For estimates of average dispersion over extended periods, the joint probability of occurrence of wind speed, wind direction, and atmospheric stability must be known. These probabilities, or frequencies, have been generated from onsite data using the vertical temperature gradient and the variability of the horizontal wind to estimate atmospheric stability in accordance with Regulatory Guide 1.23. Summaries of wind speed-wind direction-atmospheric stability joint frequencies appear in Tables 2.3-15 through 2.3-22.

The following data summarize the percent frequencies of occurrence for each stability class (determined from the temperature gradient) recorded at the Clinton Power Station site.

A	B	C	D	E	F	G
4.34	3.58	5.38	40.10	26.52	10.93	8.88

Unstable (A, B, C)	13.30%
Neutral (D)	40.10%
Stable (E, F, G)	46.33%

The combination of E stability and calm winds (< 0.3 mps) occurred 0.06% of the time; F and calm, 0.06%; G and calm, 0.12%.

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

Operation of the station will influence the local micrometeorology as a result of discharging warm water into the cooling lake. The principal meteorological effect of this will be to produce a steam fog over the lake when cold air (~41° F or less) moves over the significantly warmer (~59° F or higher) lake water. The rate of condensation of evaporated water vapor (and thus the formation of steam fog) will be greatest at the lower ambient air temperature associated with the winter months. With heavy steam fog and relatively light wind speeds (~2 meters per second - 5 mph - or less), noticeable drift of the steam fog off the lake surface is possible.

Icing caused by condensed water vapor from the lake will have a primary effect on vertical surfaces adjacent to the lake shore. Horizontal surfaces will accumulate much less rime. Observations of icing conditions from the Dresden Nuclear Power Station in Illinois indicate that icing on horizontal surfaces is not a significant problem beyond the first 200 feet from the edge of the lake.

2.3.2.2.1 Topographical Description

Figure 2.3-15 is a topographic map of the area within 50 miles of the Clinton Power Station site. Figure 2.3-16 is a topographic map of the areas within 5 miles of the site. Figure 2.3-17 shows topographic cross sections in each of the 16 compass directions radiating from the site. The crosshatched sections represent the areas to be filled in by the creation of the cooling lake. The station is located at an elevation of approximately 735 feet MSL. Within the 5-mile radius, no land elevation is above 760 feet or below 640 feet. Much of this modest relief is due to the shallow valleys surrounding the North Fork of Salt Creek and Salt Creek. These valleys form the boundaries of the Clinton Power Station cooling lake (Lake Clinton). The surface of Lake Clinton is 690 feet. Thus, a large portion of the topographical relief in the immediate area is filled by the lake.

Lake Clinton presents a discontinuity in the ground surface over which diffusing gases must travel. The lake presents a temporary smoother surface than the land over which the air parcels travel. Theoretically, this reduces the natural turbulence and thus the resulting diffusion. At the same time, however, the reduced frictional effects will allow a slight increase in the wind speed, thus adding to the rate of diffusion. In view of the relatively short travel distances across the lake for releases from the station under any wind direction, no adjustments in the diffusion calculations are proposed at this time to account for the reduction in surface roughness caused by the lake.

A more significant impact of the lake will be the warm surface it presents to the atmosphere which, during nighttime and the winter, will be significantly warmer than the surrounding ground. This increase in temperature will cause the layer of air in contact with the lake to achieve a neutral lapse rate, especially when stable conditions prevail over the land. Thus, material released from a ground-level source would receive additional diffusion in the vertical over the lake than would be computed using a stable delta T stability category determined from the meteorological tower. However, due to the dimensions of the lake and its orientation with respect to the station, no adjustments are proposed at this time to the diffusion calculations. Any additional dispersion contributed by the lake temperature effects will add to the conservatism of the accident and routine diffusion estimates.

The natural topography of the area around the site will not significantly affect the diffusion estimates.

2.3.2.2.2 <u>Prediction of Cooling Lake Steam Fog</u>

The cooling lake with once-through cooling provides a source of open water during the winter months. It is possible that cold air passing over the relatively warmer water surface can become saturated with respect to water vapor. When sufficient evaporated water vapor condenses into droplets, steam fog occurs and the transparency of the air is reduced. The characteristics of such steam fog will vary with the water temperature, the distance traveled over the water, and the low-level ambient air temperature, relative humidity, vertical and horizontal stability, and the transporting wind speed.

An analytical model was used that accounts for the processes of evaporation, condensation, and diffusion downwind and includes the variables listed previously as input conditions. A description of the model is provided in Attachment A2.3 (Analytical Fog Model).

A portion of the cooling lake will be subjected to increases in water temperature due to the operation of the station. These increases in water temperature were determined by use of the LAKET computer model. The physical characteristics of the lake, such as time-varying temperature and natural and forced evaporation, were predicted by LAKET (Transient Lake Temperature Prediction) (References 23, 24 and 25). This program simulated the effects of varying weather conditions and station heated-water discharge on the surface temperature and evaporation rates of a lake or river. The time-varying temperature distribution along the water body's central axis is computed against time, along with the natural and forced evaporation. In the case of lakes, the variation in the lake level is also computed.

Inputs to the computer program include data on the lake, the station, and the weather. Lake data include total surface area, salt content, seepage rate, initial temperature, and the length and width of the segments used in the analysis. Station data include temperature rises, flow rates, latitude, longitude, and altitude. Weather data include dates, wind speed, dry bulb temperatures, relative humidity, dew point, barometric pressure, air vapor pressure, cloud cover, and precipitation.

Output from the program provides time-varying temperature along the water body, natural and forced evaporation, and plots of temperature vs. time at nine locations.

The computational approach consists of modeling the body of water into an idealized system of prismatic volumes, each having geometric and physical characteristics (i.e., width, depth, area, and flow) unique to its location and time. Using inputted weather data, the natural water temperature is determined, and based on the station rise, the downstream temperatures are computed. The one-dimensional finite-difference procedure discussed in Reference 24 is used.

Hydraulic and thermal balances are utilized along with the energy budget method for determining the evaporation from the lake or river. The latter takes into account solar radiation, reflected solar radiation, energy transferred from the lake back to the atmosphere, and other factors.

Specific areas of interest within the immediate vicinity of the lake were defined for detailed study and evaluation of the steam fog potential and resulting impact. The seven areas selected are as follows:

a. Area 1 - road crossing the lake south of DeWitt,

b. Area 2 - the county road that runs east-west along the southern edge of the lake just west of Route 14,

c. Area 3 - Route 10 where it runs along the southern edge of the lake,

d. Area 4 - the NW-SE portion of Route 10 that is parallel to the spillway,

e. Area 5 - Route 10 and the connecting roads that run N-S along the western edge of the site,

f. Area 6 - that portion of U.S. Route 54 that is close to the lake including the bridge area over the lake, and

g. Area 7 - the reactor building complex.

Calculations showed no significant probability of the lake steam fog extending to DeWitt. Similarly, the probability of the lake steam fog reaching the town of Lane is so low that the town did not require designation as a special area. The remaining sections of roads around the lake also were not affected significantly by the predicted lake steam fog.

The steam fog prediction model described in Attachment A2.3 was used to calculate the occurrence of restricted visibility caused by steam fog in each of the specified areas. This determination required the calculation of evaporation and diffusion for each of six to ten combinations of temperature and relative humidity for each of the seven major wind directions that would affect one or more of the areas of interest. This process was repeated for each month to account for the monthly difference in water temperature. The results were several hundred maps showing the concentration of water vapor and water droplets for the lake and adjacent areas.

The time required to run the model and to evaluate the results did not permit complete variation of all the variables that would influence the horizontal extent and intensity of steam fog from the lake. Therefore, a set of values was selected and used in the model to produce what are considered the probable "worst case" for contiguous 30-day periods.

Assumptions and variables used in the model are described in Attachment A2.3. Briefly, these assumptions are as follows:

a. The wind speed shear in the layer into which water is evaporated is 1 m/sec.

b. The mean wind speed in the layer is 1 m/sec.

c. The vertical and horizontal stability in the layer is Pasquill stability category C during the period August through April.

d. The calculated lake water temperatures apply uniformly across the width of the lake.

e. The edges of the lake do not freeze and reduce the amount of surface water available for evaporation.

f. Visibility is defined by the empirical curves derived from previous fog research and presented in Attachment A2.3.

g. The horizontal visibility is measured at a height of 1 meter above the surface of the lake.

The vast majority of predicted hours of steam fog off the lake occurred when the air temperature was 5° C or less with the water temperature 10° C to 25° C or higher. These conditions would produce an "unstable" lapse rate within the layer of interest. Stability category C was selected to represent this type of stability lapse rate.

The calculated number of hours of various categories of visibility due to steam fog from the lake for each selected area are presented in Tables 2.3-23 through 2.3-29. The values in these tables are the sum of all the hours of various combinations of air temperature, relative humidity, and wind direction that could affect a given area. Thus, the values do not apply uniformly over an entire area, but rather just for that portion that is immediately downwind of the lake for the occurring wind direction.

The fog prediction model has been used to predict the hours of steam fog during the summer months. Using the same assumptions as for the other months of the year, the model predicts a greater number of hours than would be expected or is verified by the calibration data. Rather than attempt to refine the model to obtain more precise (smaller) values, the derived values are presented to serve as an upper limit on the number of hours of off-lake steam fog that could be expected.

The magnitude of lake steam fog during the summer months was not fully determined. Results showed steam fog forming during the cooler nighttime temperatures and periods of high relative humidity. Additional calibration of the model is required for these summer warm fog conditions before values can be presented.

The values presented in Tables 2.3-23 through 2.3-29 are considered to be representative of the worst probable monthly average conditions expected for the month. The basis for this conclusion is the conservative nature of the input values, described in the preceding paragraph, that were used in the model.

Normal station operating conditions (normal station operating conditions are defined as a lake elevation of 690 feet; a 70% load factor for February, March, April, May, October, and November; and an 80% load factor for June, July, August, September, and January) were used to determine the lake water temperatures. Occasional periods of heavier loads or lower lake elevations would not significantly affect the predicted steam fog hours. This conclusion is possible because continuous (365 days per year) operation under extreme station operating conditions results in a net increase in water temperature of approximately 5.5° F at the discharge and 3° F at the intake during the winter. These changes would have their greatest impact immediately below the discharge point; the impact would decrease rapidly downstream with the requirement to transport more or heavier fog farther off the lake surface into the areas of interest. Test results with slightly (2° F to 5° F) increased water temperatures showed negligible change in the resulting steam fog for a given meteorological condition.

Probably the most influential conservative factor used in the model is the assumed low-level wind speed of 1 m/sec. The impact of this assumption is indicated as follows. First, it reduces the thickness of the layer to reach saturation more rapidly and achieve a greater concentration of condensed water vapor. The low wind speed then moves the steam fog off the lake in a relatively uniform mass, albeit a shorter distance, before off-lake evaporation improves the visibility. Wind speed data collected at Clinton 10 meters above the ground during the period of record showed that only 8% of the hours had a wind speed of 1.5 m/sec or less.

Nevertheless, it is felt that wind speed of 1 m/sec should be used for a conservative approximation of the near-surface wind speed that will affect the horizontal visibility in the first 3 meters above the ground in the designated areas of interest.

The maps (not included) produced by the computer fog model show the horizontal extent of various concentrations of water vapor or condensed water that occur with a given wind direction for a specified combination of air temperature and relative humidity. Analyses of these maps show that the maximum extent of reduced visibility beyond the lake from the lake steam fog will be generally confined to the area that is south of the lake and east of the town of Lane. Steam fog can occasionally drift over U.S. Route 54 where it passes near the northern edge of the lake.

A shallow open flume about 300 feet wide will be used to carry the discharge water from the station to the discharge point in the lake approximately 3 miles due east of the station. Because of the water temperature, steaming in the flume is expected with the same frequency as for Area 1 (Table 2.3-23). However, the relative narrow width of the flume will limit the volume of air exposed to the water surface and thereby limit the amount of air to reach saturation. Under most meteorological conditions, any excess water vapor acquired over the flume will mix with the drier ambient air as soon as the parcel of air is beyond the flume. With low-level wind speeds of less than 2 m/sec, the expected extent of significantly reduced visibility due to steam fog from the flume will be limited to, at most, a few hundred feet immediately downwind of the flume. With higher wind speeds, any steam fog should dissipate within 200 feet of the flume. A greater horizontal extent will occur when the ambient air is very near saturation prior to exposure to the flume. In this case, natural fog would be expected and the steam fog from the flume would act to increase the intensity of the ambient restriction to visibility immediately downwind of the flume.

The impact of fogging and icing conditions on emergency procedures for a coincident station accident is primarily in the area of transportation. The safe speed of vehicles through the area downwind of the lake and affected by lake steam fog could be reduced if the lake steam fog is sufficiently dense. As a conservative estimate, a speed of 10 to 15 mph could still be maintained through an affected area in all but the most extreme cases.

The maximum horizontal extent of steam fog from the lake along a road is on the order of 1 mile or less. The extent of extremely dense steam fog would be limited to the road area immediately adjacent to the lake. Once vehicles are through an affected area, the speed of the vehicle is controlled by other factors.

Icing from lake steam fog should not be a problem. Roads located 500 feet or more from the lake are not expected to be affected by ice from the lake. Vertical surfaces within 500 feet downwind of the lake could accumulate rime ice under certain meteorological conditions. A horizontal surface, such as a road bed, is seldom affected by lake ice if it is 50 feet or more from the edge of the lake. If significant icing should occur on any critical road due to natural or cooling lake influences, standard highway maintenance procedures will be followed to reduce the impact of the ice on vehicle movement over the affected critical roads. The white or hoary accumulation of ice on vertical surfaces along a roadway can alert drivers and maintenance personnel to the possibility of icing conditions on the road.

2.3.2.3 Local Meteorological Conditions for Design and Operating Bases

Design and operating bases such as tornado parameters, glaze thickness, and winter probable maximum precipitation are statistics which by definition and necessity are based upon long-term regional records. While data collected at the Clinton onsite meteorological monitoring system can be considered representative of long-term site meteorology, long-term regional data are most appropriate for use as conservative estimates of climatological extremes. Therefore, all design and operating basis conditions were based upon regional meteorological data, as described in Subsection 2.3.1.2.

2.3.3 Onsite Meteorological Measurements Program

The meteorological monitoring program began at the Clinton Power Station site on April 13, 1972. The instrument systems and their locations were selected with emphasis on compliance with Regulatory Guide 1.23.

A tower with two levels of instrumentation was erected. There are no trees, tall obstructions or significant topographical features in the immediate vicinity of the tower. The ground under the tower is covered with short natural grasses and weeds. The location of the tower is shown in Figure 2.3-18.

The meteorological measurements program at the Clinton site consists of monitoring wind direction, wind speed, temperature, dewpoint, and precipitation. The Main Tower is instrumented at the 10 meter and 60 meter levels. All parameters are recorded digitally and displayed in the Main Control Room. Data recovery is expected to exceed 90% for all parameters. Two methods of determining atmospheric stability are used: delta T (vertical temperature difference) is the principal method; 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 tower is equipped with instrumentation that conforms with the system accuracy recommendations of Regulatory Guide 1.23. The equipment is placed on booms oriented into the generally prevailing wind at the site. Equipment signals are brought 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 provide estimates of airborne concentrations of gaseous effluents and projected offsite radiation dose. In addition to the meteorological instruments, an unused antenna for the Alert and Notification System is mounted on this tower at approximately 170 feet high.

Meteorological monitoring instruments have been placed on the microwave tower to act as a backup to the existing meteorological monitoring instruments on the meteorological tower.

The microwave tower is 250 feet high with instrumentation (wind speed and direction) installed at the 33-foot (10-meter) level. The current antenna for the Alert and Notification System is mounted on this tower. The location of the tower is shown on Figure 2.3-18.

The monitoring panel, located in a shelter at the base of the microwave tower, is a microprocessor based system which is used to collect, process, format and record all the meteorological data supplied. The data is displayed locally and is accessible for review and trending at the 800 foot elevation of the Control Building. Heating and air conditioning are thermostatically controlled in the shelter to provide a controlled environment for the data processing equipment.

The NRC requested an additional tape containing weather data from Clinton Power Station (1972-1979). It provided to the NRC under separate cover on September 18, 1981 (Q&R 451.01).

In response to a request for a complete record for 12 consecutive months of hour by hour onsite meteorlogical data, the following is provided (Q&R 451.02)

(a) The selected period is one year of data from 73/15/00 to 74/14/23. That is January 15, 12:00 A.M., 1973 to January 14, 11:59 P.M., 1974. (Date is YY/Julian Day/HH.)

(b) Attachment A gives the dates and hours of missing data in the selected period.

Attachment B provides recommended substitute values for the missing data.

(c) The bases for the substitutions were extrapolations and interpolation using data before and after the missing period. There were no lengthy periods of missing data which required more involved methods. There are no recommended values for precipitation given.

ATTACHMENT A
(Q&R 451.02)

List of missing data within the period 73/15/99 to 74/14/23 (Dates are YY/Julian Day/HH)

DIR = Wind Direction, Degrees

RNG = Wind Direction Variability, Degrees

SPD = Wind Speed, Meters per second

T = 10 Meter Temperature, Degrees Celsius

DT = 60m-10m Temperature, Degrees Celsius

DP = Dew Point, Degrees Celcius 10 ro 60 meter levels

P = Precipitation, inches

Date	Hours Missing	Parameter(s)
73/16/10	1	10m DIR, RNG, SPD
73/32/4	7	60m DIR, RNG, SPD
73/46/00	43	10m DIR, RNG, SPD
73/48/03	61	60m DIR, RNG, SPD
73/51/7	28	60m DIR, RNG, SPD
73/61/6	5	T, ΔT, 10 and 60m DP
73/66/1	4	10m DIR, RNG, SPD
73/94/12	1	T, ΔT, 10 and 60m DP
73/112/16	4	10 and 60m DIR, RNG, SPD, DP, T, ΔT
73/120/21	1	T, ΔT, 10 and 60m DP
73/128/8	8	P
73/135/11	2	T, ΔT, 10 and 60m DP
73/144/19	236	P
73/168/13	46	P
73/179/8	37	60m DIR, RNG, SPD
73/195/9	3	T, ΔT, 10 and 60m DP
73/197/15	2	T, ΔT, 10 and 60m DP
73/211/18	28	10m DIR, RNG, SPD
73/219/10	23	60m DIR, RNG
73/228/18	14	60m DIR, RNG, SPD
73/233/10	26	60m DIR, RNG, SPD
73/244/7	76	60m DIR, RNG
73/265/21	10	10m SPD
73/266/19	5	10m SPD
73/269/9	9	T, ΔT, 10 and 60m DP
73/318/3	32	60m DIR, RNG, SPD
73/319/22	19	60m DIR, RNG, SPD
73/320/17	4	10m SPD

ATTACHMENT A (CONT'D)
(Q&R 451.02)

Date	Hours Missing	Parameter(s)
73/320/17	91	P
73/333/11	4	60m DIR, RNG, SPD
73/335/19	41	60m DIR, RNG, SPD
73/346/1	10	60m SPD
73/358/21	18	10m DIR, ENG
73/361/3	7	10m SPD
73/10/19	15	10m SPD

ATTACHMENT B
(Q&R 451.02)

Substitute values for missing data within period from 73/15/00 to 74/14/23 (Dates are YY/Julian Day/HH)

DIR	=	Wind Direction, Degrees
RNG	=	Wind Direction, Variability, Degrees
SPD	=	Wind Speed, Meters per second
T	=	10 Meter Temperature, Degrees Celsius
∆T	=	60m-10m Temperature, Degrees Celsius
DP	=	Dew Point, Degrees Celcius 10 or 60 meter levels
P	=	Precipitation, inches

Date	Hour		Parameter			Hour	Parameter		
73/16		10M	DIR.	RNG.	SPD.				
			179.	50.	80.				
73/32		60M	DIR.	RNG.	SPD.				
	4		137.	55.	10.0				
	5		131.	52.	10.1				
	6		118.	53.	10.3				
	7		128.	60.	10.5				
	8		135.	56.	10.4				
	9		138.	50.	11.0				
	10		145.	52.	10.9				
73/46		10M	DIR.	RNG.	SPD.		DIR.	RNG.	SPD
	0		327.	57.	4.4	11	320.	63.	7.4
	1		320.	60.	4.0	12	317.	43.	7.4
	2		323.	50.	5.0	13	315.	57.	6.5
	3		329.	52.	6.2	14	312.	64.	7.1
	4		330.	55.	7.0	15	316.	57.	5.7
	5		330.	52.	6.2	16	312.	60.	6.0
	6		325.	48.	7.0	17	310.	60.	5.3
	7		322.	47.	8.2	18	311.	58.	6.0
	8		325.	55.	8.1	19	320.	44.	5.4
	9		325.	52.	8.2	20	340.	48.	6.0
	10		319.	62.	6.8				
73/46	21		330.	45.	5.8	8	331.	59.	6.4
	22		329.	47.	5.2	9	327.	68.	6.2
	23		315.	65.	4.6	10	327.	59.	6.1
	0		305.	54.	5.0	11	328.	72.	5.8
	1		308.	49.	5.0	12	326.	67.	6.1
	2		316.	62.	4.8	13	330.	61.	6.5

Date	Hour		Parameter			Hour	Parameter		
	3		309.	56.	4.3	14	332.	62.	6.3
	4		311.	54.	5.8	15	334.	57.	5.9
	5		315.	54.	5.4	16	332.	55.	5.8
	6		320.	37.	6.0	17	335.	36.	5.7
	7		330.	46.	6.3	18	337.	39.	5.4
73/48		60M	DIR.	RNG.	SPD		DIR.	RNG.	SPD
	3		45.	60.	3.2	10	194.	70.	7.2
	4		219.	59.	2.8	11	193.	76.	7.5
	5		268.	9.	2.8	12	189.	63.	7.0
	6		271.	39.	4.2	13	194.	56.	7.0
	7		277.	35.	3.3	14	202.	69.	7.3
	8		255.	61.	2.8	15	204.	76.	6.9
	9		155.	105.	4.7	16	200.	54.	7.1
	10		163.	95.	5.4	17	195.	57.	7.0
	11		178.	87.	6.7	18	199.	54.	5.6
	12		168.	87.	6.9	19	187.	59.	5.6
	13		179.	89.	7.4	20	189.	67.	7.3
	14		176.	90.	6.8	21	198.	51.	6.0
	15		180.	91.	6.5	22	216.	53.	5.5
	16		179.	77.	6.2	23	209.	47.	5.7
	17		185.	92.	6.3	0	205.	57.	4.2
	18		177.	82.	6.4	1	210.	50.	4.1
	19		173.	65.	6.6	2	201.	42.	5.1
	20		177.	52.	6.4	3	210.	41.	5.4
	21		179.	52.	6.5	4	216.	44.	5.8
	22		181.	61.	7.0	5	204.	54.	6.1
	23		181.	61.	6.0	6	222.	58.	5.5
	0		183.	65.	6.1	7	206.	41.	5.5
	1		199.	62.	6.0	8	211.	122.	5.1
	2		206.	55.	7.1	9	208.	61.	5.2
	3		207.	59.	7.2	10	230.	85.	5.5
	4		201.	56.	6.2	11	236.	63.	6.3
	5		187.	53.	5.7	12	228.	66.	7.2
	6		186.	52.	6.1	13	216.	78.	8.2
	7		183.	63.	6.3	14	220.	74.	7.8
	8		181.	52.	6.0	15	226.	67.	9.4
	9		184.	58.	5.9				
73/51/7		60M	DIR.	RNG.	SPD		DIR.	RNG.	SPD
	7		297.	67.	10.4	0	193.	65.	6.7
	8		298.	76.	10.1	1	206.	73.	8.9
	9		296.	56.	10.2	2	232.	62.	9.2
	10		296.	66.	9.6	3	250.	37.	9.6
	11		295.	62.	9.7	4	272.	54.	11.1
	12		305.	78.	8.9	5	288.	44.	10.4
	13		296.	79.	8.4	6	295.	55.	9.3
	14		283.	72.	8.6	7	299.	62.	8.9

Date	Hour		Parameter			Hour	Parameter		
	15		285.	79.	8.2	8	307.	68.	10.2
	16		278.	61.	6.7	9	304.	67.	11.2
	17		265.	43.	6.8	10	306.	53.	11.9
	18		249.	41.	5.9				
	19		238.	43.	4.5				
	20		199.	35.	4.7				
	21		189.	46.	5.4				
	22		193.	44.	5.7				
	23		194.	64.	6.4				
	Hour	T	ΔT	10MDP	60MDP				
73/61									
	6	7.6		3.6	3.0				
	7	7.8	.30	4.0	3.1				
	8	8.0	-.09	5.0	3.5				
	9	8.1	-.10	6.0	4.8				
	10	8.2	-.14	6.8	5.8				
	Hour		10M	DIR.	RNG.				
73/66									
	1		208.	69.					
	2		198.	67.					
	3		202.	66.					
	4		211.	80.					
	Hour	T	ΔT	10MDP	60MDP				
73/94									
	12	3.8	-0.42	1.4	1.1				
	Hour	10M	DIR.	RNG.	SPD				
73/112									
	16		250.	45.	2.0				
	17		251.	55.	1.8				
	18		249.	48.	2.2				
	19		252.	52.	2.1				
		60M	DIR.	RNG.	SPD				
			260.	30.	3.8				
			255.	35.	3.5				
			257.	45.	4.1				
			262.	40.	4.3				
73/112		10MT	T	10MDP	60MDP	P			
	16	16.1	0.20	8.8	8.0	0.00			
	17	16.0	0.45	8.7	7.9	0.00			
	18	15.8	0.75	8.6	7.4	0.00			
	19	15.7	0.80	8.4	7.3	0.00			
73/120			.T	ΔT	10MDP	60MDP			
	21		19.0	-0.20	14.1	14.6			

Date	Hour		Parameter			Hour	Parameter		
	8		Precip		8 hr.	No DATA			
73/128									
73/135			.T	ΔT	10MDP	60MDP			
	11		13.8	-0.85	-4.2	-4.0			
	12		14.6	-0.87	-4.5	-4.0			
73/144/19 thru									
73/152/15			Precip		236 hr.	No DATA			
73/168/13 thru									
73/170/12			Precip		46 hr.	No DATA			
73/179	Hour	60M	DIR.	RNG.	SPD	Hour	DIR.	RNG.	SPD
	8		304.	58.	9.4	3	303.	78.	3.1
	9		310.	53.	9.4	4	325.	36.	3.5
	10		301.	60.	9.5	5	341.	108.	2.5
	11		321.	71.	9.1	6	281.	68.	2.9
	12		311.	73.	9.2	7	344.	73.	3.1
	13		300.	71.	6.0	8	345.	131.	2.4
	14		301.	51.	10.5	9	317.	65.	6.3
	15		298.	69.	9.7	10	305.	74.	5.0
	16		301.	55.	5.7	11	316.	134.	4.5
	17		305.	59.	9.1	12	302.	96.	6.7
	18		305.	51.	8.0	13	289.	100.	7.4
	19		306.	48.	5.4	14	272.	80.	5.8
	20		299.	51.	4.2	15	289.	40.	7.5
	21		283.	34.	3.6	16	278.	75.	6.0
	22		246.	29.	3.4	17	289.	54.	7.0
	23		246.	28.	3.8	18	282.	49.	5.7
	0		257.	20.	4.5	19	290.	104.	3.4
	1		275.	32.	3.6	20	275.	32.	3.1
	2		272.	37.	2.7				
73/195	Hour		T	ΔT		10MDP		60MDP	
	9		24.0	-0.65		16.8		17.2	
	10		25.0	-0.70		17.0		17.4	
	11		26.0	-0.75		17.1		17.6	
73/197	Hour		T	ΔT		10MDP		60MDP	
	15		26.3	-0.74		8.4		8.4	
	16		26.2	-0.70		8.2		7.9	
73/211									
		10M	DIR.	RNG.	SPD		DIR.	RNG.	SPD
	18		328.	79.	0.4	8	326.	55.	2.0
	19		154.	69.	3.3	9	328.	83.	1.6

Date	Hour		Parameter			Hour	Parameter		
	20		145.	44.	3.2	10	349.	63.	2.0
	21		188.	57.	3.7	11	3.	53.	1.9
	22		198.	59.	0.4	12	345.	70.	3.9
	23		170.	42.	1.7	13	351.	68.	4.3
	0		149.	70.	0.3	14	346.	67.	5.4
	1		192.	30.	0.4	15	356.	53.	6.8
	2		149.	14.	0.4	16	360.	55.	7.0
	3		181.	0.	0.6	17	4.	45.	3.8
	4		246.	19.	0.6	18	8.	42.	4.8
	5		286.	0.	0.7	19	351.	39.	4.1
	6		292.	25.	0.8	20	335.	39.	4.5
	7		304.	59.	0.7	21	346.	60.	4.5
73/219	Hour	60M	DIR.	RNG.			DIR.	RNG.	
	10		201.	79.		22	172.	68.	
	11		200.	87.		23	170.	60.	
	12		187.	72.		0	183.	68.	
	13		192.	76.		1	182.	74.	
	14		191.	94.		2	183.	70.	
	15		194.	76.		3	187.	71.	
	16		188.	60.		4	193.	62.	
	17		179.	71.		5	190.	53.	
	18		168.	62.		6	188.	56.	
	19		130.	50.		7	193.	54.	
	20		144.	52.		8	195.	77.	
	21		147.	55.					
73/228	Hour	60M	DIR.	RNG.	SPD		DIR.	RNG.	SPD
	18		145.	50.	2.3	1	128.	44.	2.3
	19		127.	26.	2.4	2	115.	34.	2.0
	20		125.	46.	1.2	3	104.	53.	2.1
	21		127.	67.	1.8	4	109.	47.	2.5
	22		103.	59.	2.1	5	135.	28.	2.7
	23		113.	61.	2.2	6	127.	29.	2.8
	0		144.	52.	2.2	7	120.	51.	2.9
73/233	Hour	60M	DIR.	RNG.	SPD		DIR.	RNG.	SPD
	10		44.	81.	5.6	23	60.	53.	4.0
	11		10.	138.	4.9	0	33.	27.	4.0
	12		14.	142.	5.1	1	4.	46.	4.8
	13		31.	137.	5.7	2	51.	58.	4.6
	14		10.	91.	5.9	3	71.	65.	4.7
	15		4.	61.	6.9	4	83.	96.	4.3
	16		13.	69.	6.7	5	78.	79.	3.4
	17		343.	47.	4.9	6	75.	65.	3.5
	18		342.	59.	4.0	7	90.	67.	5.4
	19		51.	93.	4.1	8	117.	73.	6.5
	20		59.	77.	3.9	9	129.	94.	7.0

ATTACHMENT B (CONT'D)
(Q&R 451.02)

Date	Hour		Parameter			Hour	Parameter		
	21		79.	69.	5.2	10	140.	95.	6.9
	22		56.	45.	3.8	11	103.	101.	5.5
73/244	Hour								
		60M	DIR.	RNG.	SPD		DIR.	RNG.	SPD
	7		125.	69.		9	175.	86.	
	8		157.	72.		10	182.	85.	
	9		171.	72.		11	179.	91.	
	10		172.	92.		12	176.	84.	
	11		186.	106.		13	178.	81.	
	12		173.	82.		14	172.	88.	
	13		182.	93.		15	176.	72.	
	14		176.	83.		16	175.	86.	
	15		166.	87.		17	177.	69.	
	16		164.	86.		18	159.	63.	
	17		148.	65.		19	142.	41.	
	18		148.	71.		20	130.	49.	
	19		150.	65.		21	128.	47.	
	20		146.	60.		22	136.	53.	
	21		153.	60.		23	131.	44.	
	22		154.	72.		0	180.	83.	
	23		159.	72.		1	138.	60.	
	0		164.	66.		2	140.	27.	
	1		166.	63.		3	150.	27.	
	2		145.	55.		4	136.	24.	
	3		142.	55.		5	147.	28.	
	4		138.	50.		6	159.	63.	
	5		157.	80.		7	149.	71.	
	6		153.	69.		8	146.	71.	
	7		145.	86.		9	161.	89.	
	8		167.	85.		10	167.	88.	
	11		175.	87.		23	158.	90.	
	12		217.	66.		0	145.	32.	
	13		200.	73.		1	161.	64.	
	14		169.	71.		2	199.	53.	
	15		179.	88.		3	205.	139.	
	16		186.	60.		4	209.	102.	
	17		184.	79.		5	204.	67.	
	18		179.	73.		6	192.	55.	
	19		174.	76.		7	179.	65.	
	20		197.	69.		8	210.	75.	
	21		182.	74.		9	218.	77.	
	22		200.	66.		10	223.	71.	
73/265									
	Hour	10M	SPD						
	21		3.0						

ATTACHMENT B (CONT'D)
(Q&R 451.02)

Date	Hour		Parameter			Hour	Parameter		
	22		3.2						
	23		2.9						
	0		2.5						
	1		2.0						
	2		1.3						
	3		0.8						
	4		1.0						
	5		1.0						
	6		0.8						
73/266									
	Hour	10M							
			DIR.	SPD			DIR.	SPD	
	19			2.4		22		4.5	
	20			2.8		23		5.0	
	21			4.0					
73/269									
	Hour	T	ΔT		10MDP	60MDP			
	9	22.8	-0.48		17.3	16.6			
	10	23.0	-0.42		17.+	16.7			
	11	23.1	-0.52		17.4	16.8			
	12	23.3	-0.45		17.5	16.9			
	13	23.4	-0.38		17.5	16.9			
	14	23.6	-0.45		17.6	16.9			
	15	23.8	-0.10		17.6	17.0			
	16	24.2	-0.59		17.6	17.1			
	17	24.8	-0.85		17.6	17.3			
73/318									
	Hour	60M	DIR.	RNG.	SPD	Hour	DIR.	RNG.	SPD
	3		243.	163.	2.0	19	194.	46.	10.6
	4		200.	63.	3.2	20	191.	60.	10.4
	5		186.	54.	3.5	21	195.	53.	10.5
	6		153.	53.	3.2	22	192.	59.	10.7
	7		154.	55.	3.0	23	197.	53.	11.1
	8		149.	94.	3.4	0	192.	50.	10.6
	9		129.	60.	5.2	1	196.	44.	9.8
	10		158.	72.	5.4	2	206.	56.	9.6
	11		162.	86.	7.0	3	198.	45.	8.8
	12		155.	76.	7.1	4	194.	60.	9.7
	13		163.	66.	7.6	5	207.	64.	7.3
	14		170.	71.	8.2	6	257.	94.	8.1
	15		169.	64.	8.5	7	180.	33.	5.1
	16		176.	54.	8.8	8	213.	65.	7.5
	17		182.	58.	9.8	9	261.	35.	11.5
	18		188.	50.	10.1	10	262.	55.	11.7
73/319	Hour	60M	DIR.	RNG.	SPD	Hour	DIR.	RNG.	SPD
	22		311.	51.	5.5	8	269.	30.	4.0

Date	Hour		Parameter			Hour	Parameter		
	23		285.	55.	4.5	9	298.	66.	5.1
	0		262.	34.	4.6	10	293.	62.	5.0
	1		291.	49.	7.4	11	289.	63.	5.0
	2		311.	46.	8.1	12	283.	76.	5.0
	3		316.	39.	8.3	13	284.	56.	5.1
	4		308.	46.	6.0	14	264.	108.	4.3
	5		298.	45.	5.0	15	247.	77.	3.5
	6		300.	60.	4.0	16	210.	105.	3.0
	7		251.	37.	3.0				
73/320	Hour	10M	SPD			Hour	SPD		
	17		0.6			19	0.4		
	18		0.4			20	1.5		
73/320/17	-	73/324/10	Precip	No DATA					
73/333	Hour	60M	DIR.	RNG.	SPD	Hour	DIR.	RNG.	SPD
	11		198.	80.	2.8	13	207.	55.	6.3
	12		197.	94.	4.2	14	199.	63.	5.8
73/335		60M	DIR.	RNG.	SPD		DIR.	RNG.	SPD
	19		104.	51.	6.4	16	163.	64.	6.6
	20		110.	46.	7.6	17	167.	58.	6.7
	21		120.	49.	7.0	18	176.	53.	6.2
	22		115.	44.	7.2	19	175.	61.	6.4
	23		124.	42.	6.1	20	181.	51.	8.8
	0		132.	41.	5.9	21	182.	50.	8.8
	1		139.	39.	6.1	22	193.	49.	8.7
	2		145.	38.	5.6	23	186.	48.	9.5
	3		140.	37.	5.9	0	200.	43.	8.2
	4		150.	44.	6.3	1	198.	49.	7.6
	5		152.	42.	6.6	2	189.	53.	7.4
	6		153.	38.	5.5	3	189.	50.	6.9
	7		152.	38.	5.0	4	185.	40.	7.0
	8		162.	56.	5.7	5	183.	36.	7.1
	9		168.	60.	6.6	6	182.	43.	7.2
	10		182.	50.	7.3	7	185.	38.	7.2
	11		176.	73.	7.5	8	186.	50.	6.4
	12		177.	62.	8.0	9	190.	43.	7.3
	13		177.	63.	9.5	10	195.	54.	8.1
	14		175.	66.	9.1	11	200.	48.	8.3
	15		167.	64.	8.5				
73/346	Hour	60M	SPD			Hour	SPD		
	1		5.8			6	2.2		
	2		4.7			7	2.5		
	3		2.2			8	1.5		
	4		3.5			9	3.1		
	5		3.3			10	3.0		

Date	Hour		Parameter		Hour	Parameter	
73/358	Hour	10M	DIR.	RNG	Hour	DIR.	RNG.
	21		116.	37.	8	172.	30.
	22		116.	50.	9	157.	44.
	23		119.	51.	10	176.	50.
	0		127.	39.	11	185.	42.
	1		136.	47.	12	214.	57.
	2		149.	47.			
	3		151.	48.			
	4		175.	26.			
	5		181.	40.			
	6		174.	25.			
	7		172.	27.			
73/361							
	Hour	10M	SPD		Hour	SPD	
	3		3.4		7	3.4	
	4		3.6		8	3.5	
	5		3.5		9	4.0	
	6		3.8				
74/10							
	Hour	10M	SPD		Hour	SPD	
	19		3.6		3	3.4	
	20		3.2		4	4.1	
	21		3.2		5	3.8	
	22		2.6		6	3.7	
	23		3.0		7	2.0	
	0		3.1		8	2.2	
	1		3.7		9	3.0	
	2		3.6				

2.3.4 Short-Term Diffusion Estimates

2.3.4.1 Objective

Conservative estimates of the local atmospheric dilution factors (X/Q) and their 5% probability level conditions for the Clinton Power Station site have been prepared for the exclusion area boundary (EAB), actual site boundary (ASB), and distances of 0.5, 1.5, 2.5, 3.5, 4.5, 7.5, 15, 25, 35, and 45 miles. Calculations were made for sliding time period windows of 1, 8, 16, 72, and 624 hours from onsite meteorological data for the period May 1972 through April 1977.

2.3.4.2 Calculations

Calculations of ground-level atmospheric dilution factors for the CPS site were performed using Gaussian plume diffusion models for a continuously emitting ground level source. Hourly centerline X/Q values were computed from the concurrent hourly mean values of wind speed, wind direction and range, and Pasquill stability class of the onsite meteorological data. The wind speed at the 10 meter level was used in the diffusion estimates for the ground-level release. The Pasquill stability class was determined from the measured vertical temperature difference (ΔT) and the variance of the horizontal wind field ($\delta\theta$) according to Regulatory Guide 1.23. Calms were assigned a wind speed value equal to the starting speed of the wind vane (0.7 mph). Cumulative frequency distributions were prepared to determine the χ/Q values that were exceeded 5% and 50% of the time.

2.3.4.3 Atmospheric Diffusion Models and Frequency Distributions

Gaussian plume diffusion models for ground-level concentration were used to describe the downwind spread of effluents for the Clinton Power Station. A continuous ground-level release of effluents at a constant emission rate was assumed in the diffusion estimates. Total reflection of the plume at ground-level was assumed in the diffusion estimates: i.e., there is no deposition or reaction at the surface. Hourly X/Q values were calculated by the following equations:

$$\frac{X}{Q} = \frac{1}{u_{10}\pi\Sigma_y\sigma_z} \qquad (2.3\text{-}1)$$

$$\frac{X}{Q} = \frac{1}{u_{10}(\pi\sigma_y\sigma_z + A/2)} \qquad (2.3\text{-}2)$$

or

$$\frac{X}{Q} = \frac{1}{u_{10} \quad (3\pi\sigma_y\sigma_z)} \qquad (2.3\text{-}3)$$

where

$\frac{X}{Q}$ is the relative centerline concentration (sec/m^3) at ground level

π is 3.14159

u_{10} is the wind speed (m/sec) at 10 meters above the ground

Σ_y is the lateral plume spread (m), a function of atmospheric stability, wind speed, and downwind distance from the point of release. For distances to 800 meters, $\Sigma_y = M\sigma_y$; M being a function of atmospheric stability and wind speed. For distances greater than 800 meters, Sy = (M-1) sy800m + sy

σ_y is the lateral plume spread as a function of atmospheric stability and distance

σ_z is the vertical plume spread as a function of atmospheric stability and distance.

A is the smallest vertical plane, cross-sectional area of the building from which the effluent is released (A=2069m^2).

For neutral to stable conditions with wind speeds less than 6m/sec Equations 2.3-2 and 2.3-3 were calculated and compared, and the higher X/Q was selected. This higher value was compared to the X/Q resulting from Equation 2.3-1 and the lower was selected. This was done in accordance with Regulatory Guide 1.145, Atmospheric Dispersion Models For Potential Accident Consequence Assessments At Nuclear Power Plants. For all other stability and/or wind speed conditions, X/Q was selected as the higher value from Equations 2.3-2 and 2.3-3.

From these hourly X/Q values, cumulative frequency distributions were prepared from the mean values of sliding time windows of 1, 2, 8, 16, 72, and 624 hours. These intervals correspond to time periods of 0-1 hour, 0-2 hours, 0-8 hours, 8-24 hours, 1-4 daysand 4-30 days. For each time period used, the mean centerline X/Q value in each sector was computed.

The results are presented in Tables 2.3-30 through 2.3-43.

2.3.5 Short-Term (Accident) Diffusion Estimates (Alternative Source Term X/Q Analysis)

2.3.5.1 Objective

Estimates of atmospheric diffusion (X/Q) at the Exclusion Area Boundary (EAB), the outer boundary of the Low Population Zone (LPZ) and the Control Room Intakes have been prepared 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. Calculations were made based on onsite meteorological data for the years 2000 through 2002.

2.3.5.2 Calculation of X/Q at the EAB and LPZ

X/Q was calculated at the EAB (975 m) and LPZ (4018 m) for the Standby Gas Treatment/HVAC Vent Stack using the NRC-recommended model PAVAN (Reference 27).

This stack does not qualify as an elevated release as defined by Regulatory Guide 1.145 (Reference 28); therefore, it is executed as a "ground" type release.

X/Q values at the EAB and LPZ were calculated by PAVAN in accordance with Regulatory Guide 1.145. For ground-level releases, calculation for the 2 hours following the accident is based on the following equations:

$$\chi/Q = \frac{1}{\overline{U}_{10}\left(\pi\sigma_y\sigma_z + A/2\right)} \qquad (2.3.5\text{-}1)$$

$$\chi/Q = \frac{1}{\overline{U}_{10}\left(3\pi\sigma_y\sigma_z\right)} \qquad (2.3.5\text{-}2)$$

$$\chi/Q = \frac{1}{\overline{U}_{10}\pi\Sigma_y\sigma_z} \qquad (2.3.5\text{-}3)$$

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 with meander and building wake effects (in meters), a function of atmospheric stability, wind speed, and distance [for distances of 800 m or less, Σ_y = Mσ_y, where M is determined from Reg. Guide 1.145 Fig. 3; for distances greater than 800 m, Σ_y =(M-1) σ_y 800 m + σ_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 2.3.5-1 and 2.3.5-2 is compared to the value of Equation 2.3.5-3 for meander, and the lower value is selected. For all other conditions (stability classes A, B, or C), meander is not considered and the highest X/Q value of equations 2.3.5-1 and 2.3.5-2 is selected.

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

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

Determination by PAVAN of the LPZ maximum sector X/Q is based on a logarithmic interpolation between the 2-hour sector X/Q and the annual average X/Q for the same sector. For each time period, the highest of these 16 sector X/Q values is identified as the maximum sector X/Q value. The maximum sector X/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 X/Q.

The “5% overall site X/Q” values for the EAB and LPZ are each determined by constructing an overall cumulative probability distribution for all directions. The 0-2 hour X/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 X/Q value that is exceeded 5% of the time is determined. PAVAN then calculates the 5% overall site X/Q at the LPZ for intermediate time periods by logarithmic interpolation of the maximum of the 16 annual average X/Q values and the 5% 2-hour X/Q values.

2.3.5.2.1 PAVAN Meteorological Databases

The meteorological database to be utilized for the EAB and LPZ X/Q calculations were prepared for use in PAVAN by transforming the three years (i.e. 2000-2002) of hourly meteorological tower data observations into a joint wind speed-wind direction-stability class occurrence frequency distribution as shown in Tables 2.3-45 and 2.3-46. In accordance with Regulatory Guide 1.145, atmospheric stability class was determined by vertical temperature difference between the 60 m and the 10-m 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 29) with the first category identified as "calm". The higher of the starting speeds of the wind vane and anemometer (i.e. 0.50 mph) was used as the threshold for calm winds, per Regulatory Guide 1.145, Section 1.1. 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:

PAVAN WIND SPEED CATEGORIES

Category No.	Regulatory Guide 1.23 Speed Interval (mph)	Pavan-Assumed Speed Interval (mph)
1 (Calm)	0 to < 1	0 to <0.50
2	1 to 3	>=0.50 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

The procedures used by PAVAN assign 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.

2.3.5.2.2 PAVAN Model Input Parameters

The Standby Gas Treatment/HVAC Vent Stack has height of 60.5 m above station grade, however since it does not qualify as an elevated release per Regulatory Guide 1.145, PAVAN requires that its height be assigned an input value of 10 m. For this assumed non-elevated stack scenario, EAB and LPZ receptor terrain elevation is not considered.

The smallest Control Building vertical projected area of the 1093.5 m^2 was utilized (h=33.5 m, w=32.6 m; based on drawing M01-1115.

2.3.5.2.3 PAVAN EAB and LPZ X/Q Modeling Results

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

OFFSITE X/Q SUMMARY (sec/m^3) Standby Gas Treatment Vent / HVAC Vent Stack					
Receptor	0-2 hour	2-8 hour	8-24 hour	1-4 day	4-30 day
EAB (975 m)	2.46E-04	1.19E-04	8.30E-05	3.78E-05	1.22E-05
LPZ (4018 m)	5.62E-05	2.48E-05	1.65E-05	6.81E-06	1.91E-06

2.3.5.3 Calculation of X/Q at the Control Room Intake

Estimates of atmospheric diffusion (X/Q) are made at each of the three Control Room Intakes (i.e. East, West, and Normal) for releases from the Standby Gas Treatment Vent/HVAC Vent Stack for periods of 2, 8, and 16 hours and for 3 and 26 days. The NRC-sponsored computer code ARCON96 (Reference 30), consistent with the procedures in Draft Regulatory Guide DG-1111 (Reference 31) is utilized.

2.3.5.3.1 ARCON96 Model Analysis

Since the Standby Gas Treatment/HVAC Vent Stack is not 2.5 times the height of the adjacent structures, it does not qualify as an elevated release per DG-1111, therefore, ARCON96 is executed in vent release mode. With an assumed zero (0) vertical exit velocity, vent releases are treated as ground-level releases by ARCON96. The basic model for a ground-level release is

$$\frac{\chi}{Q} = \frac{1}{\pi \sigma_y \sigma_z U} \exp\left[-0.5\left(\frac{y}{\sigma_y}\right)^2\right] \tag{2.3.5-4}$$

where:

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

σ_y, σ_z = diffusion coefficients (m)

U = wind speed (m/s)

y = distance from the center 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 32) codes is used for σ_y and σ_z.

The diffusion coefficients have the 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. The parameters are defined for 3 distance ranges – 0 to 100 m, 100 to 1000 m, and greater than 1000 m. The 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 incorporated as follows:

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 by

$$\Sigma_y = \left(\sigma_y{}^2 + \Delta\sigma_{y1}{}^2 + \Delta\sigma_{y2}{}^2\right)^{1/2} \tag{2.3.5-5}$$

$$\Sigma_z = \left(\sigma_z{}^2 + \Delta\sigma_{z1}{}^2 + \Delta\sigma_{z2}{}^2\right)^{1/2} \tag{2.3.5-6}$$

The variables σ_y and σ_z are the normal 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 Ramsdall and Fosmire (Reference 33). The low wind speed corrections are:

$$\Delta\sigma_{y1}{}^{2} = 9.13\times10^{5}\left[1-\left(1+\frac{x}{1000U}\right)\exp\left(\frac{-x}{1000U}\right)\right] \tag{2.3.5-7}$$

$$\Delta\sigma_{z1}{}^{2} = 6.67\times10^{2}\left[1-\left(1+\frac{x}{100U}\right)\exp\left(\frac{-x}{100U}\right)\right] \tag{2.3.5-8}$$

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 coefficients corrections that account for enhanced diffusion in the wake have a similar form. These corrections are:

$$\Delta\sigma_{y2}{}^{2} = 5.24\times10^{-2}\,U^{2}A\left[1-\left(1+\frac{x}{10\sqrt{A}}\right)\exp\left(\frac{-x}{10\sqrt{A}}\right)\right] \tag{2.3.5-9}$$

$$\Delta\sigma_{z2}{}^{2} = 1.17\times10^{-2}\,U^{2}A\left[1-\left(1+\frac{x}{10\sqrt{A}}\right)\exp\left(\frac{-x}{10\sqrt{A}}\right)\right] \tag{2.3.5-10}$$

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

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

$$\Sigma_{ymax} = \frac{2\pi x}{\sqrt{12}} \qquad \approx 1.81x \tag{2.3.5-11}$$

2.3.5.3.1.1 ARCON96 Meteorological Databases

The 2000-2002 meteorological databases utilized in ARCON96 consists of hourly meteorological data observations of wind speed and direction, and delta temperature stability class.

The designation of 'calm' is made to all wind speed observations of less than 0.5 mph. The higher of the starting speeds of the Climatronics® wind vane and anemometer equipment on each of the towers (i.e. 0.50 mph) was used as the threshold for calm winds, per Regulatory Guide 1.145, Section 1.1.

2.3.5.3.1.2 ARCON96 Input Parameters

The parameters that were input into the ARCON96 model for use in calculating the Control Room X/Q are summarized below:

ARCON96 MODEL INPUT PARAMETERS

ARCON96 INPUT PARAMETER	East Intake	West Intake	Normal Intake
Release Height (m)	60.5	60.5	60.5
Intake Height (m)	29.9	18.6	28.3
Horizontal Distance from Intake to Stack (m)	69.8	69.8	51.5
Elevation Difference between Stack Grade and Intake Grade (m)	0	0	0
Building Area (m^2)	1093.5	1093.5	1093.5
Direction from Intake To Stack (°)	288	168	260
Vertical Velocity (m/s)	0	0	0
Stack Flow (m^3/s)	0	0	0
Stack Radius (m)	0	0	0

2.3.5.3.1.3 ARCON96 Control Room X/Q Results

A summary of the atmospheric diffusion estimates at the Control Room Intakes for releases from the Standby Gas Treatment/HVAC Vent Stack is shown below.

ARCON96 Control Room Intake X/Q Results (sec/m^3)
Standby Gas Treatment/HVAC Vent Stack

INTAKE	0-2 Hour	2-8 Hour	8-24 Hour	1-4 Day	4-30 Day
East Intake	9.75E-04	7.09E-04	2.93E-04	2.13E-04	1.79E-04
West Intake	9.45E-04	7.58E-04	3.28E-04	2.61E-04	1.85E-04
Normal Intake *	1.54E-03	1.09E-03	4.67E-04	3.21E-04	2.64E-04

* (maximum intake X/Q value)

2.3.6 Long-Term (Routine) Diffusion Estimates

2.3.6.1 Objective

Annual average dilution factors were computed for routine releases from the common station vent along the side of the containment building. The MESODIF model was used. Meteorological data observed on the tower at the Clinton site were used. The period of record was May 14, 1972 through April 30, 1977.

2.3.6.2 Calculations

MESODIF employs an integrated puff model concept. This model differs from ordinary Gaussian type models in that it will allow released materials to be transported back over the source in the event of a wind shift. MESODIF carries the effluent as a string of puffs released into the wind field as observed by the onsite meteorological station. Individual puffs are tracked until they are either too dilute to be of further significance or else leave the area being considered. The integrated puff concept yields a conservative estimate of concentration near the source. Ground level releases were assumed in order to yield conservative estimates.

The MESODIF program developed by the Air Resources Laboratories (ARL) personnel at Idaho Falls, Idaho was described by Start and Wendell (Reference 26). A program source deck was obtained from ARL in January 1978. Modifications were made to the program to accommodate input data from a single site rather than a number of stations as used in Idaho. The modifications are described in the following paragraphs.

Subroutines RNGRD9 and ASCND were deleted from the program. Subroutine ASCND was used to move elements of an array. Subroutine RNGRD9 was used to read the wind direction and speed data, convert the direction and speed to U- and V-components and to interpolate the components from station locations to a grid array.

Meteorological data for stability and mixing depth were read in the main program. The array in the main program has space available for wind direction and speed so these data were supplied there and conversion to U- and V-components was accomplished in the main program.

Wind direction was provided to the nearest degree and wind speed to the nearest mile per hour. Conversion to U- and V-components was accomplished by:

$$\theta = (270 - WD)\,\pi/180$$

$$U = S \cos\theta$$

$$V = S \sin\theta$$

where WD is wind direction and S is wind speed for any hour. The U- and V-components calculated in this manner were assigned to each grid point. Two U, V arrays are carried in the program because an interpolation is performed to account for changes with time. This modification maintains both arrays at two times.

A test case supplied by ARL was run before and after the change. Constant wind direction and speeds were assumed at all stations for the "before" run. Identical results were achieved for the test runs.

Hourly data from May 11, 1972 through April 30, 1977 were used. Integrated dosages were calculated for each year and the hourly values averaged for the five year period. The rectangular array of points from 2 mile and 10 mile grids were plotted. Sector centerline values were derived from the data and plotted on log-log graph paper. A straight line was drawn through the points and the relative concentrations were read at the required distances. Actual model calculations were made at distances ranging from two miles to 45 miles from the source. Data are listed in Table 2.3-44 for the period of record.

2.3.7 References

1. L. Denmark, "Climates of the States: Illinois," No. 60-11, U.S. Department of Commerce, August 1959 (Revised June 1969).

2. "Local Climatological Data, Annual Summary with Comparative Data, Peoria, Illinois, 1976," U.S. Department of Commerce, NOAA, Asheville, North Carolina.

3. "Local Climatological Data, Annual Summary with Comparative Data, Springfield, Illinois, 1976," U.S. Department of Commerce, NOAA, Asheville, North Carolina.

4. R. A. Bryson, "Airmasses, Streamlines and the Boreal Forest," Technical Report No. 24, pp. 13-57, University of Wisconsin: Dept. of Meteorology, Madison, Wisconsin, 1966.

5. S. A. Changnon, Jr., "Climatology of Hourly Occurrences of Selected Atmospheric Phenomena in Illinois," Circular 93, Illinois State Water Survey, Urbana, Illinois, 1968.

6. S. A. Changnon, Jr., Thunderstorm-Precipitation Relations in Illinois, Report of Investigation No. 34, Illinois State Water Survey, Urbana, Illinois, 1957.

7. Glossary of Meteorology, (Edited by R. E. Huschke), Second Printing with Corrections, American Meteorological Society, Boston, Massachusetts, 1970.

8. "Severe Local Storm Occurrences, 1955-1967," WBTM FCST 12, U.S. Department of Commerce, ESSA, Silver Spring, Maryland, September 1969.

9. F. A. Huff and S. A. Changnon, Jr., Hail Climatology of Illinois, Report of Investigation 38, Illinois State Water Survey, Urbana, Illinois, 1959.

10. J. L. Marshall, "Probability of a Lightning Stroke," Lightning Protection, Chapter 3, pp. 30-31, John Wiley and Sons, New York, New York, 1971.

11. J. W. Wilson and S. A. Changnon, Jr., "Illinois Tornadoes," Circular 103, Illinois State Water Survey, Urbana, Illinois, 1971.

12. J. C. S. Thom, "Tornado Probabilities," Monthly Weather Review, Volume 91, pp. 730-736, 1963.

13. "Design Basis Tornado for Nuclear Power Plants," Regulatory Guide 1.76, U.S. Atomic Energy Commission, April 1974.

14. "Building Code Requirements for Minimum Design Loads in Buildings and Other Structures," ANSI A58.1-1972, American National Standards Institute, Inc., New York, New York, 1972.

15. S. A. Changnon, Jr., "Climatology of Severe Winter Storms in Illinois," Bulletin 53, Illinois State Water Survey, Urbana, Illinois, 1969.

16. "Glaze Storm Loading Summary, 1927-28 to 1936-37," Association of American Railroads, 1955.

17. "Snow Load Studies," Paper 19, Housing and Home Finance Agency, Division of Housing Research, and U.S. Weather Bureau, Washington, D.C., 1952.

18. J. T. Riedel, J. F. Appleby, and R. W. Schloemer, "Seasonal Variation of the Probable Maximum Precipitation East of the 105th Meridian for Areas from 10 to 1000 Square Miles and Durations of 6, 12, 24, and 48 Hours," HMR No. 33, U.S. Department of Commerce, Washington, D.C., April 1956.

19. C. R. Hosler, "Low-Level Inversion Frequency in the Contiguous United States," Monthly Weather Review, Volume 89, pp. 319-339, September 1961.

20. G. C. Holzworth, "Mixing Heights, Wind Speeds, and Potential for Urban Air Pollution Throughout the Contiguous United States," AP-101, U.S. Environmental Protection Agency, Office of Air Programs, Research Triangle Park, North Carolina, January 1972.

21. R. E. Huschke (Ed.), "Glossary of Meteorology," American Meteorological Society, Boston, Massachusetts, 1959 (Second Printing with Corrections, 1970).

22. U.S. Department of Commerce, NOAA, Environmental Data Service, "Local Climatological Data, Annual Summary with Comparative Data," Springfield and Peoria, Illinois (two documents).

23. Texas Water Development Board, "Simulation of Water Quality in Streams and Canals," Report 128, May 1971.

24. "Prediction of Thermal Energy Distribution in Streams and Reservoirs," Water Resources Engineers, Inc., Prepared for the Department of Fish and Game, State of California, August 30, 1968.

25. G. E. Harbeck, et al., "Water-Loss Investigations: Lake Mead Studies," Geological Survey Professional Paper 298, U.S. Govt. Printing Office, Washington, D.C., 1958.

26. G. E. Start and L. L. Wendell, "Regional Effluent Dispersion Calculations Considering Spatial and Temporal Meteorological Variations," NDAA Technical Memorandum ERL-ARL-44, Air Resources Laboratories, Idaho Falls, Idaho, 1974.

27. Atmospheric Dispersion Code System for Evaluating Accidental Radioactivity Releases from Nuclear Power Stations; PAVAN, Version 2; Oak Ridge National Laboratory; U.S. Nuclear Regulatory Commission; December 1997.

28. Regulatory Guide 1.145; Atmospheric Dispersion Models for Potential Accident Consequence Assessments at Nuclear Power Plants (Revison 1); U.S. Nuclear Regulatory Commission; November 1982.

29. Regulatory Guide 1.23 (Safety Guide 23), Onsite Meteorological Programs; U. S. Nuclear Regulatory Commission; USNRC Office of Standards Development; Washington, D.C.; 1972.

30. Atmospheric Relative Concentrations in Building Wakes; NUREG/CR-6331, PNNL-10521, Rev. 1; prepared by J. V. Ramsdell, Jr., C. A. Simmons, Pacific Northwest National Laboratory; prepared for U.S. Nuclear Regulatory Commission; May 1997 (Errata, July 1997).

31. Draft Regulatory Guide DG-1111; Atmospheric Relative Concentrations for Control Room Radiological Habitability Assessments at Nuclear Power Plants; U.S. Nuclear Regulatory Commission; December 2001.

32. XOQDOQ: Computer Program for the Meteorological Evaluation of Routine Releases at Nuclear Power Stations; NUREG/CR-2919; J. F. Sagendorf, J. T. Goll, and W. F. Sandusky, U.S. Nuclear Regulatory Commission; Washington, D.C; 1982.

33. Atmospheric Dispersion Estimates in the Vicinity of Buildings; J. V. Ramsdell and C. J. Fosmire, Pacific Northwest Laboratory; 1995.

TABLE 2.3-1
CLIMATOLOGICAL DATA FROM WEATHER STATIONS SURROUNDING
CLINTON POWER STATION

PARAMETER	STATION	
	PEORIA	SPRINGFIELD
Temperature (°F)		
Annual average	50.8	52.7
Maximum	103 (July 1940)	112 (July 1954)
Minimum	-20 (Jan. 1963)	-22 (Feb. 1963)
Degree days	6098	5558
Relative Humidity (%)		
Annual average at:		
6 a.m.	83	82
12 noon	62	60
Wind		
Annual average speed (mph)	10.3	11.4
Prevailing Direction	S	S
Fastest mile:		
Speed (mph)	75 (July	75 (June
Direction	NW 1953)	SW 1957)
Precipitation (in.)		
Annual average	35.06	35.02
Monthly maximum	13.09 (Sept. 1961)	9.91 (Apr. 1964)
Monthly minimum	0.03 (Oct. 1964)	0.15 (Dec. 1955)
24-hour maximum	5.06 (Apr. 1950	5.12 (Sept. 1959)
Snowfall (in.)		
Annual average	23.4	22.3
Monthly maximum	18.9 (Dec. 1973)	22.7 (Dec. 1973)
Maximum 24-hour	10.2 (Dec. 1973)	10.9 (Dec. 1973)
Mean Annual (no. of days)		
Precipitation ≥ 0.1 in	111	112
Snow, sleet, hail ≤ 1.0 in.	8	8
Thunderstorms	49	50
Heavy fog (visibility 1/4mile or less)	21	18
Maximum temperature ≥ 90° F	17	28
Minimum temperature ≤ 32° F	132	119

* The data presented in this table are based upon References 2 and 3. These statistics are based on periods of record ranging from 17 to 39 years in length. The ranges span the years 1937 to 1976.

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 (No. 12) WIRE	CITY	STATE SECTION
2-4 Feb. 1883			11	Springfield	WSW
20 Mar. 1912	0.5			Decatur	C
21 Feb. 1913	2.0			La Salle	NE
12 Mar. 1923	1.6		12	Marengo	NE
17-19 Dec. 1924	1.2	15:1	8	Springfield	WSW
22-23 Jan. 1927	1.1		2	Cairo	SE
31 Mar. 1929	0.5			Moline	NW
7-8 Jan. 1930	1.2			Carlinville	WSW
1-2 Mar. 1932	0.5			Galena	NW
7-8 Jan. 1937	1.5			Quincy	W
31 Dec. 1947 - 1 Jan. 1948	1.0		72	Chicago	NE
10 Jan. 1949	0.8			Macomb	W
8 Dec. 1956				Alton	WSW
20-22 Jan. 1959	0.7	12:1		Urbana	E
26-27 Jan. 1967	1.7	17:1	40	Urbana	E

NOTE: 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

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

TABLE 2.3-4
10M TEMPERATURE (DEG. C)

	AVERAGE DAILY	AVERAGE DAILY MAXIMUM	AVERAGE DAILY MINIMUM	ABSOLUTE MAXIMUM	ABSOLUTE MINIMUM
January	-5.1	-1.3	-8.9	15.5	-28.8
February	-1.3	1.9	-4.4	15.8	-23.6
March	5.9	10.5	1.6	25.5	-15.1
April	11.4	16.7	6.1	29.3	-6.5
May	16.4	21.2	11.2	32.1	0.0
June	21.2	26.1	16.0	33.0	5.0
July	23.6	28.4	18.5	35.2	8.1
August	22.1	26.8	17.4	23.2	9.1
September	17.7	22.8	12.7	33.3	0.8
October	11.9	17.1	6.9	30.0	-4.8
November	4.5	8.4	0.8	23.0	-15.8
December	-2.3	1.3	-5.9	17.8	-23.8
Period of Record	10.5	15.0	6.0	35.2	-28.8

TABLE 2.3-5
HOURS WITH ^{x}C TEMPERATURE

	32.2 DEG. OR MORE		0.0 DEG. OR LESS		-12.2 DEG. OR LESS.		-17.8 DEG. OR LESS	
	hr.	%	hr.	%	hr	%	hr.	%
January	(0)	0.0	(2628)	72.5	(730)	20.1	(225)	6.2
February	(0)	0.0	(2019)	60.5	(203)	6.1	(48)	1.4
March	(0)	0.0	(808)	21.9	(19)	0.5	(0)	0.0
April	(0)	0.0	(188)	4.7	(0)	0.0	(0)	0.0
May	(0)	0.0	(1)	0.0	(0)	0.0	(0)	0.0
June	(8)	0.2	(0)	0.0	(0)	0.0	(0)	0.0
July	(67)	1.9	(0)	0.0	(0)	0.0	(0)	0.0
August	(0)	0.0	(0)	0.0	(0)	0.0	(0)	0.0
September	(3)	0.1	(0)	0.0	(0)	0.0	(0)	0.0
October	(0)	0.0	(82)	2.3	(0)	0.0	(0)	0.0
November	(0)	0.0	(948)	26.4	(28)	0.8	(0)	0.0
December	(0)	0.0	(2414)	65.9	(302)	8.2	(56)	1.5
Period of Record	(78)	0.2	(9088)	21.0	(1282)	3.0	(329)	0.8

TABLE 2.3-6
DAYS WITH ^{x}C TEMPERATURE

	32.2 DEG. OR MORE		0.0 DEG. OR LESS		-12.2 DEG. OR LESS		-17.8 DEG. OR LESS	
	DAYS	%	DAYS	%	DAYS	%	DAYS	%
January	(0)	0.0	(132)	86.3	(55)	35.9	(24)	15.7
February	(0)	0.0	(116)	82.3	(21)	14.9	(6)	4.3
March	(0)	0.0	(65)	41.9	(2)	1.3	(0)	0.0
April	(0)	0.0	(27)	16.2	(0)	0.0	(0)	0.0
May	(0)	0.0	(1)	0.6	(0)	0.0	(0)	0.0
June	(3)	2.0	(0)	0.0	(0)	0.0	(0)	0.0
July	(15)	10.0	(0)	0.0	(0)	0.0	(0)	0.0
August	(0)	0.0	(0)	0.0	(0)	0.0	(0)	0.0
September	(1)	0.7	(0)	0.0	(0)	0.0	(0)	0.0
October	(0)	0.0	(15)	9.9	(0)	0.0	(0)	0.0
November	(0)	0.0	(73)	48.7	(3)	2.0	(0)	0.0
December	(0)	0.0	(129)	83.8	(29)	18.8	(8)	5.2
Period of Record	(19)	1.0	(558)	30.5	(110)	6.0	(38)	2.1

TABLE 2.3-7
CLINTON POWER STATION SITE RELATIVE HUMIDITY SUMMARY
COMPOSITE

	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	PERIOD OF RECORD
Average	85.94	82.04	77.29	68.01	64.44	68.24	70.00	74.04	72.15	67.15	77.58	85.71	68.28
Average Daily Max.	92.10	89.77	87.75	83.96	80.77	83.26	85.13	86.04	85.33	80.75	86.61	90.47	79.01
Average Daily Min.	71.04	65.71	56.91	46.43	43.89	47.52	49.03	53.84	49.40	45.57	60.44	71.64	50.63
Absolute Max.	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00
Absolute Min.	38.34	14.11	22.26	16.80	15.78	19.22	27.20	23.93	15.91	14.86	23.13	21.40	14.11
Average for Hours													
00	83.15	80.78	74.30	69.75	68.25	70.72	69.96	76.71	73.91	67.56	76.45	82.07	68.35
03	84.00	81.27	75.53	74.31	73.88	75.17	75.54	80.02	78.10	71.51	78.10	82.49	71.15
06	84.88	82.23	79.17	77.55	75.88	76.23	77.75	82.62	80.27	74.87	79.87	83.10	73.04
09	84.31	79.85	71.60	66.35	61.19	64.77	66.22	73.67	73.38	68.40	77.39	82.10	66.35
12	78.10	75.28	63.31	54.95	52.41	53.97	55.67	61.81	59.77	56.74	67.48	77.51	57.85
15	74.32	71.11	59.83	53.07	49.43	50.32	50.25	56.39	51.12	49.93	63.62	74.12	53.79
18	78.53	75.99	64.18	54.48	52.14	52.18	54.35	61.51	56.89	53.79	69.04	79.07	57.52
21	81.66	78.76	63.76	63.76	61.91	61.11	65.27	70.98	67.38	62.08	74.42	81.32	64.26

TABLE 2.3-8
CLINTON POWER STATION SITE WET BULB TEMPERATURE SUMMARY
°C

	COMPOSITE												
	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	PERIOD OF RECORD
Average	-4.13	0.43	8.67	13.74	18.84	23.19	25.49	23.64	19.95	14.10	6.23	-1.22	11.34
Average Daily Max.	0.27	4.31	13.36	19.21	23.52	27.32	29.28	27.61	24.79	19.38	10.42	2.65	15.41
Average Daily Min.	-7.98	-3.16	3.60	7.16	12.27	16.69	19.10	17.67	13.75	7.95	1.80	-5.00	6.33
Absolute Max.	16.67	17.76	27.70	32.13	33.00	34.17	35.59	32.41	33.15	33.13	24.67	18.67	35.59
Absolute Min.	-28.35	-20.42	-13.70	-6.05	2.25	5.52	9.64	9.64	1.00	-4.16	14.95	-23.32	-28.35
57% Wet Bulb Value													28.17

TABLE 2.3-9
33 FOOT DEW POINT

	AVERAGE DAILY	AVERAGE DAILY MAXIMUM	AVERAGE DAILY MINIMUM	ABSOLUTE MAXIMUM	ABSOLUTE MINIMUM
January	-7.8	-4.4	-11.1	14.1	-29.5
February	-4.0	-0.7	-7.5	13.6	-24.1
March	1.8	5.4	-1.2	17.7	-17.8
April	4.2	7.4	1.3	19.0	-10.0
May	8.1	11.0	5.2	22.7	-9.0
June	13.5	16.4	10.6	25.6	-0.3
July	16.5	19.3	14.0	25.	3.5
August	15.9	18.1	13.6	24.5	2.5
September	11.4	14.0	8.5	23.3	-7.1
October	4.2	7.1	1.4	9.1	-11.3
November	-0.1	2.8	-2.7	16.3	-17.5
December	-5.2	-2.1	-8.3	13.1	-25.7
Period of Record	4.7	7.8	1.9	25.6	-29.5

TABLE 2.3-10
PERCENT OF HOURS WITH DEW POINT

	18.3 DEG. C OR MORE	12.8 DEG. C OR MORE	7.2 DEG. C OR MORE	0.0 DEG. C OR MORE
January	0.0	0.1	2.0	16.5
February	0.0	0.2	3.5	27.9
March	0.0	5.9	21.7	58.9
April	0.1	9.9	32.8	73.7
May	3.0	22.1	59.1	89.5
June	19.3	54.1	89.0	99.9
July	38.1	79.3	98.1	100.0
August	37.7	73.9	94.3	100.0
September	20.3	41.1	73.0	96.2
October	0.4	13.5	34.1	72.5
November	0.0	4.6	15.0	47.3
December	0.0	0.1	2.5	17.9
Period of Record	9.5	24.9	43.3	66.3

TABLE 2.3-11
PERCENT OF HOURS WITH DEW POINT SPREAD

	0.0 to 0.7 DEG. C	0.8 TO 2.2 DEG. C	2.3 TO 4.4 DEG. C	4.5 or DEG. C
January	15.8	33.0	37.3	14.0
February	20.1	20.7	26.8	32.3
March	6.6	18.0	29.0	46.5
April	3.4	14.2	21.1	61.2
May	1.4	9.0	22.7	66.9
June	3.0	11.1	20.5	65.4
July	2.6	8.3	22.0	67.1
August	3.0	16.3	25.9	54.8
September	5.0	16.8	23.5	54.7
October	4.5	14.9	16.2	64.4
November	7.6	20.8	31.1	40.6
December	12.7	26.7	31.8	18.8
Period of Record	7.0	18.4	25.8	48.8

TABLE 2.3-12
PRECIPITATION (INCHES)

	TOTAL	MAXIMUM FOR 1 HR.	MAXIMUM IN. 1 DAY	% HRS. WITH PRECIPITATION 0.01 IN. OR MORE	% HRS. WITH PRECIPITATION 1.00 IN. OR MORE	% DAYS WITH PRECIPITATION 0.01 OR MORE	% DAYS WITH PRECIPITATION 1.00 OR MORE	MAX. CONSECUTIVE HRS. WITH PRECIP.	MAX. CONSECUTIVE HRS. WITHOUT PRECIP.	MAX. CONSECUTIVE DAYS WITH PRECIP.	MAX. CONSECUTIVE DAYS WITHOUT PRECIP.
January	7.00	0.50	2.53	3.4	0.0	21.3	0.6	14	356	5	14
February	5.74	0.26	0.97	3.3	0.0	19.9	0.0	9	470	3	19
March	17.21	0.69	1.29	5.9	0.0	23.3	1.9	10	408	3	16
April	9.16	0.69	1.63	3.4	0.0	25.1	0.6	14	455	5	18
May	8.98	0.52	0.62	3.6	0.0	26.0	0.0	6	293	5	12
June	20.80	1.15	2.72	4.7	0.0	31.3	3.3	14	545	5	22
July	11.34	0.43	1.74	3.1	0.0	25.2	0.6	7	365	4	14
August	12.59	0.80	1.34	2.9	0.0	21.9	0.6	8	476	3	21
September	12.20	0.81	1.26	3.8	0.0	28.0	2.0	11	372	8	15
October	7.64	0.45	0.94	3.7	0.0	20.6	0.0	12	332	3	13
November	9.13	0.40	1.06	4.4	0.0	22.0	0.7	11	620	5	25
December	6.67	0.34	0.93	3.7	0.0	21.9	0.0	8	406	8	16
Period of Record	128.45	1.15	2.72	3.8	0.0	24.6	0.9	14	807	8	33

TABLE 2.3-13
MONTHLY FREQUENCY OF FOG OCCURRENCE, HOURS OF MAXIMUM AND MINIMUM, AND FOG PERSISTENCE FOR PEORIA, ILLINOIS (1949-1951; 1957-1971)

						NUMBER OF TIMES IN 15 YEARS FOG PERSISTED FOR AT LEAST		
	TOTAL FREQUENCY	DAILY MAXIMUM		DAILY MINIMUM				
MONTH	OF OCCURRENCES (%)	HOUR	%	HOUR	%	12 HOURS	24 HOURS	MAX.
Jan.	17.8	8AM	25.1	6PM	14.0	38	15	95
Feb.	17.1	8AM	26.8	3PM	11.6	32	8	42
March	14.9	6AM	24.1	3PM	9.5	33	8	74
April	8.2	6AM	18.0	2PM	4.1	10	4	36
May	7.4	6AM	17.2	5PM	2.5	11	2	34
June	5.7	5AM	17.4	6PM	0.9	3	1	42
July	7.3	5AM	27.6	5PM	0.7	7	0	15
Aug.	8.6	6AM	35.7	4PM	0.4	5	0	19
Sept.	9.1	6AM	27.3	2PM	1.9	10	1	33
Oct.	10.3	7AM	23.3	3PM	5.4	15	3	34
Nov.	13.8	8AM	23.0	1PM	8.5	25	7	43
Dec.	15.5	9AM	21.5	4PM	10.0	38	9	48

TABLE 2.3-14
MONTHLY FREQUENCY OF FOG OCCURRENCE, HOURS OF MAXIMUM AND MINIMUM, AND FOG PERSISTENCE FOR SPRINGFIELD, ILLINOIS (1951-1961; 1963-1970)

						NUMBER OF TIMES IN 15 YEARS FOG PERSISTED FOR AT LEAST		
	TOTAL FREQUENCY	DAILY MAXIMUM		DAILY MINIMUM				
MONTH	OF OCCURRENCES (%)	HOUR	%	HOUR	%	12 HOURS	24 HOURS	MAX.
January	17.2	7AM	25.1	3PM	13.4	49	17	90
February	15.0	7AM	23.9	3PM	10.8	39	15	53
March	12.7	6AM	21.4	3PM	8.7	36	8	36
April	6.4	6AM	16.1	4PM	2.3	16	2	26
May	5.5	5AM	14.6	4PM	1.5	8	1	27
June	3.7	6AM	12.4	5PM	0.8	1	1	29
July	5.0	5AM	22.3	3PM	0.2	6	0	19
August	6.1	6AM	27.0	4PM	0.2	2	0	13
September	5.5	6AM	23.9	4PM	0.3	3	0	22
October	6.7	6AM	15.8	4PM	4.0	14	3	47
November	9.4	7AM	17.4	2PM	4.9	25	5	51
December	15.4	8AM	20.8	2PM	12.2	37	17	75

TABLE 2.3-15
JOINT FREQUENCY DISTRIBUTION

CLINTON POWER STATION

33 FT WIND DISTRIBUTION OF WIND DIRECTIONS AND SPEEDS 4/14/72 - 4/30/77

198-33 FT DELTA T STABILITY A - DELTA T LESS THAN -1.8 DEG C PER 100 METERS

	DIRECTION																
SPEED (MPS)	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	N	TOTAL
0.3- 1.4	1	4	3	2	2	7	9	5	5	6	2	3	4	3	4	5	65
(1)	0.06	0.23	0.17	0.11	0.11	0.40	0.51	0.28	0.28	0.34	0.11	0.17	0.23	0.17	0.23	0.28	3.68
(2)	0.00	0.01	0.01	0.00	0.00	0.02	0.02	0.01	0.01	0.01	0.00	0.01	0.01	0.01	0.01	0.05	0.16
1.5- 3.0	23	24	12	14	8	19	34	41	31	37	13	24	30	27	18	24	379
(1)	1.30	1.36	0.68	0.79	0.45	1.08	1.93	2.32	1.76	2.10	0.74	1.36	1.70	1.53	1.02	1.36	21.46
(2)	0.06	0.06	0.03	0.03	0.02	0.05	0.08	0.10	0.08	0.09	0.03	0.06	0.07	0.07	0.04	0.06	0.93
3.1- 5.0	39	43	26	19	8	17	38	61	40	65	32	44	37	57	24	29	579
(1)	2.21	2.43	1.47	1.08	0.45	0.96	2.15	3.45	2.27	3.68	1.81	2.49	2.10	3.23	1.36	1.64	32.79
(2)	0.10	0.11	0.06	0.05	0.02	0.04	0.09	0.15	0.10	0.16	0.08	0.11	0.09	0.14	0.06	0.07	1.42
5.1- 8.0	28	59	27	8	4	10	22	46	38	52	46	71	65	48	49	26	594
(1)	1.59	3.34	1.25	0.45	0.23	0.57	1.25	2.60	2.15	2.94	2.60	4.02	3.68	2.72	2.77	1.47	33.64
(2)	0.07	0.15	0.05	0.02	0.01	0.02	0.05	0.11	0.09	0.13	0.11	0.17	0.16	0.12	0.12	0.06	1.46
8.1-10.4	4	2	2	0	0	0	1	9	6	11	13	19	8	5	13	6	104
(1)	0.23	0.11	0.11	0.00	0.00	0.00	0.06	0.51	0.34	0.62	1.02	1.08	0.45	0.28	0.74	0.34	5.89
(2)	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.02	0.01	0.03	0.04	0.05	0.02	0.01	0.03	0.01	0.26
OVER 10.4	0	12	1	1	2	0	1	0	2	2	3	7	2	4	2	5	44
(1)	0.00	0.68	0.06	0.06	0.11	0.00	0.06	0.00	0.11	0.11	0.17	0.40	0.11	0.23	0.11	0.28	2.49
(2)	0.00	0.03	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.02	0.00	0.01	0.00	0.01	0.11
ALL SPEEDS	95	144	66	44	24	53	105	162	122	173	114	168	146	144	110	95	1765
(1)	5.38	8.15	3.74	2.49	1.36	3.00	5.95	9.17	6.91	9.80	6.46	9.51	8.27	8.15	6.23	5.38	99.94
(2)	0.23	0.35	0.16	0.11	0.06	0.13	0.26	0.40	0.30	0.43	0.28	0.41	0.36	0.35	0.27	0.23	4.34

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(1) = PERCENT OF ALL GOOD OB S FOR THIS PAGE

(2) = PERCENT OF ALL GOOD OB S FOR THE PERIOD

1766 HRS ON THIS PAGE 1 HRS (0.1 PCT) LESS THAN 0.3 MPS (0.0 PCT OF ALL HRS)

TABLE 2.3-16
JOINT FREQUENCY DISTRIBUTION

CLINTON POWER STATION
33 FT WIND DISTRIBUTION OF WIND DIRECTIONS AND SPEEDS 4/14/72 - 4/30/77
198-33 FT DELTA T STABILITY B - DELTA T -1.8 TO -1.7 DEG C PER 100 METERS

	DIRECTION																
SPEED (MPS)	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	N	TOTAL
0.3- 1.4	0	4	5	1	0	1	1	2	1	6	2	5	4	2	2	0	36
(1)	0.00	0.27	0.34	0.07	0.00	0.07	0.07	0.14	0.07	0.41	0.14	0.34	0.27	0.14	0.14	0.00	2.47
(2)	0.00	0.01	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.00	0.01	0.01	0.00	0.00	0.00	0.09
1.5- 3.0	12	24	8	13	10	10	14	22	13	36	22	15	18	15	13	15	260
(1)	0.82	1.65	0.55	0.89	0.69	0.69	0.96	1.51	0.69	2.47	1.51	1.03	1.24	1.03	0.89	1.03	17.86
(2)	0.03	0.06	0.02	0.03	0.02	0.02	0.03	0.05	0.03	0.09	0.05	0.04	0.04	0.04	0.03	0.04	0.64
3.1- 5.0	35	32	18	14	17	24	29	41	45	61	40	46	40	43	28	27	541
(1)	2.40	2.20	1.24	0.96	1.17	1.72	1.99	2.82	3.09	4.19	2.75	3.16	2.75	2.95	1.92	1.85	37.16
(2)	0.09	0.08	0.04	0.03	0.04	0.06	0.07	0.10	0.11	0.15	0.10	0.11	0.10	0.11	0.07	0.07	1.33
5.1- 8.0	20	34	16	20	6	16	31	27	35	46	42	40	47	47	22	26	475
(1)	1.37	2.34	1.10	1.37	0.41	1.10	2.13	1.85	2.40	3.16	2.88	2.76	3.23	3.23	1.51	1.79	32.62
(2)	0.05	0.08	0.04	0.05	0.01	0.04	0.08	0.07	0.09	0.11	0.10	0.10	0.12	0.12	0.05	0.06	1.17
8.1-10.4	3	0	0	1	0	0	2	7	5	5	9	24	16	4	3	3	82
(1)	0.21	0.00	0.00	0.07	0.00	0.00	0.14	0.48	0.34	0.34	0.62	1.65	1.10	0.27	0.21	0.21	5.63
(2)	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.02	0.01	0.01	0.02	0.06	0.04	0.01	0.01	0.01	0.20
OVER 10.4	2	1	0	2	6	2	1	6	3	4	5	8	15	1	0	5	61
(1)	0.14	0.07	0.00	0.14	0.41	0.14	0.07	0.41	0.21	0.27	0.34	0.55	1.03	0.07	0.00	0.34	4.19
(2)	0.00	0.00	0.00	0.00	0.01	0.00	0.00	0.01	0.01	0.01	0.01	0.02	0.04	0.00	0.00	0.01	0.15
ALL SPEEDS	72	95	47	51	39	54	78	105	102	158	120	138	140	112	68	76	1455
(1)	4.95	6.52	3.23	3.50	2.68	3.71	5.36	7.21	7.01	10.85	8.24	9.48	9.62	7.69	4.67	5.22	99.93
(2)	0.18	0.23	0.12	0.13	0.10	0.13	0.19	0.26	0.25	0.39	0.30	0.34	0.34	0.28	0.17	0.19	3.58

(1) = PERCENT OF ALL GOOD OB S FOR THIS PAGE

(2) = PERCENT OF ALL GOOD OB S FOR THE PERIOD

1456 HRS ON THIS PAGE 1 HRS (0.1 PCT) LESS THAN 0.3 MPS (0.0 PCT OF ALL HRS)

TABLE 2.3-17
JOINT FREQUENCY DISTRIBUTION

CLINTON POWER STATION
33 FT WIND DISTRIBUTION OF WIND DIRECTIONS AND SPEEDS 4/14/72 - 4/30/77
198-33 FT DELTA T STABILITY C - DELTA T -1.6 TO -1.5 DEG C PER 100 METERS

	DIRECTION																
SPEED (MPS)	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	N	TOTAL
0.3 - 1.4	0	5	4	1	1	3	7	7	7	4	5	5	6	4	3	2	64
(1)	0.00	0.23	0.18	0.05	0.05	0.14	0.32	0.32	0.32	0.18	0.23	0.23	0.27	0.18	0.14	0.09	2.92
(2)	0.00	0.01	0.01	0.00	0.00	0.01	0.02	0.02	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.00	0.16
1.5- 3.0	27	31	31	18	12	25	29	36	29	32	22	28	35	18	28	22	423
(1)	1.23	1.42	1.42	0.82	0.55	1.14	1.32	1.64	1.32	1.46	1.01	1.28	1.60	0.82	1.28	1.01	19.32
(2)	0.07	0.08	0.08	0.04	0.03	0.06	0.07	0.09	0.07	0.08	0.05	0.07	0.09	0.04	0.07	0.05	1.04
3.1- 5.0	42	46	40	31	31	24	51	55	47	83	67	38	62	50	52	27	746
(1)	1.92	2.10	1.83	1.42	1.42	1.10	2.33	2.51	2.15	3.79	3.06	1.74	2.83	2.28	2.38	1.23	34.08
(2)	0.10	0.11	0.10	0.08	0.08	0.06	0.13	0.14	0.12	0.20	0.16	0.09	0.15	0.12	0.13	0.07	1.83
5.1- 8.0	35	34	19	20	20	31	40	33	43	88	62	61	72	55	33	29	675
(1)	1.60	1.55	0.87	0.91	0.91	1.42	1.83	1.51	1.96	4.02	2.83	2.79	3.29	2.51	1.51	1.32	30.84
(2)	0.09	0.08	0.05	0.05	0.05	0.08	0.10	0.08	0.11	0.22	0.15	0.15	0.18	0.14	0.08	0.07	1.66
8.1-10.4	8	3	0	1	0	2	2	9	14	12	17	36	20	13	5	7	149
(1)	0.37	0.14	0.00	0.05	0.00	0.09	0.09	0.41	0.64	0.55	0.78	1.64	0.91	0.59	0.23	0.32	6.81
(2)	0.02	0.01	0.00	0.00	0.00	0.00	0.00	0.02	0.03	0.03	0.04	0.09	0.05	0.03	0.01	0.02	0.37
OVER 10.4	1	3	1	8	7	9	10	3	12	9	19	23	12	4	4	5	130
(1)	0.05	0.14	0.05	0.37	0.32	0.41	0.46	0.14	0.55	0.41	0.87	1.05	0.55	0.18	0.18	0.23	5.94
(2)	0.00	0.01	0.00	0.02	0.02	0.02	0.02	0.01	0.03	0.02	0.05	0.06	0.03	0.01	0.01	0.01	0.32
ALL SPEEDS	113	122	95	79	71	94	139	143	152	228	192	191	207	144	125	92	2187
(1)	5.16	5.57	4.34	3.61	3.24	4.29	6.35	6.53	6.94	10.42	8.77	8.73	9.46	6.58	5.71	4.20	99.91
(2)	0.28	0.30	0.23	0.19	0.17	0.23	0.34	0.35	0.37	0.56	0.47	0.47	0.51	0.35	0.31	0.23	5.38

(1) = PERCENT OF ALL GOOD OB S FOR THIS PAGE

(2) = PERCENT OF ALL GOOD OB S FOR THE PERIOD

2189 HRS ON THIS PAGE 2 HRS (0.1 PCT) LESS THAN 0.3 MPS (0.0 PCT OF ALL HRS)

TABLE 2.3-18
JOINT FREQUENCY DISTRIBUTION

CLINTON POWER STATION
33 FT WIND DISTRIBUTION OF WIND DIRECTIONS AND SPEEDS 4/14/72 - 4/30/77
198-33 FT DELTA T STABILITY D - DELTA T -1.4 TO -0.5 DEG C PER 100 METERS

	DIRECTION																
SPEED (MPS)	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	N	TOTAL
0.3 - 1.4	30	34	31	37	40	25	46	50	46	52	37	36	46	26	35	31	602
(1)	0.18	0.21	0.19	0.23	0.25	0.15	0.28	0.31	0.28	0.32	0.23	0.22	0.28	0.16	0.21	0.19	3.69
(2)	0.07	0.08	0.08	0.09	0.10	0.06	0.11	0.12	0.11	0.13	0.09	0.09	0.11	0.06	0.09	0.08	1.48
1.5- 3.0	126	178	204	197	147	173	250	249	218	229	160	162	190	166	155	135	2939
(1)	0.77	1.09	1.25	1.21	0.90	1.06	1.53	1.53	1.34	1.40	0.98	0.99	1.16	1.02	0.95	0.83	18.01
(2)	0.31	0.44	0.50	0.48	0.36	0.43	0.61	0.61	0.54	0.56	0.39	0.40	0.47	0.41	0.38	0.33	7.23
3.1- 5.0	269	289	291	286	248	231	302	416	466	396	314	360	450	406	316	294	5334
(1)	1.65	1.77	1.78	1.75	1.52	1.42	1.85	2.55	2.86	2.43	1.92	2.21	2.76	2.49	1.94	1.80	32.69
(2)	0.66	0.71	0.72	0.70	0.61	0.57	0.74	1.02	1.15	0.97	0.77	0.89	1.11	1.00	0.78	0.72	13.11
5.1- 8.0	240	263	138	134	170	193	228	439	515	428	323	535	679	457	319	269	5330
(1)	1.47	1.61	0.85	0.82	1.04	1.18	1.40	2.69	3.16	2.62	1.98	3.28	4.16	2.80	1.96	1.65	32.67
(2)	0.59	0.65	0.34	0.33	0.42	0.47	0.56	1.08	1.27	1.05	0.79	1.32	1.67	1.12	0.78	0.66	13.10
8.1-10.4	65	63	11	16	16	23	40	152	139	119	137	200	204	102	86	73	1446
(1)	0.40	0.39	0.07	0.10	0.10	0.14	0.25	0.93	0.85	0.73	0.84	1.23	1.25	0.63	0.53	0.85	8.86
(2)	0.16	0.15	0.03	0.04	0.04	0.06	0.10	0.37	0.34	0.29	0.34	0.42	0.50	0.25	0.21	0.18	3.55
OVER 10.4	25	19	13	21	18	22	17	39	58	52	95	132	80	24	24	23	662
(1)	0.15	0.12	0.08	0.13	0.11	0.13	0.10	0.24	0.36	0.32	0.58	0.81	0.49	0.15	0.15	0.14	4.06
(2)	0.06	0.05	0.03	0.05	0.04	0.05	0.04	0.10	0.14	0.13	0.23	0.32	0.20	0.06	0.06	0.06	1.63
ALL SPEEDS	755	846	688	691	639	667	883	1345	1442	1276	1066	1425	1649	1181	935	825	16313
(1)	4.63	5.18	4.22	4.23	3.92	4.09	5.41	8.24	8.84	7.82	6.53	8.73	10.11	7.24	5.73	5.06	99.98
(2)	1.86	2.08	1.69	1.70	1.57	1.64	26.17	3.31	3.55	3.14	2.62	3.50	4.05	2.90	2.30	2.03	40.10

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(1) = PERCENT OF ALL GOOD OB S FOR THIS PAGE

(2) = PERCENT OF ALL GOOD OB S FOR THE PERIOD

16317 HRS ON THIS PAGE 4 HRS (0.0 PCT) LESS THAN 0.3 MPS (0.0 PCT OF ALL HRS)

TABLE 2.3-19
JOINT FREQUENCY DISTRIBUTION

CLINTON POWER STATION
33 FT WIND DISTRIBUTION OF WIND DIRECTIONS AND SPEEDS 4/14/72 - 4/30/77

198-33 FT DELTA T STABILITY E - DELTA T -0.4 TO +1.5 DEG C PER 100 METERS

	DIRECTION																
SPEED (MPS)	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	N	TOTAL
0.3 - 1.4	38	42	49	47	33	53	62	69	60	60	48	44	41	28	19	32	725
(1)	0.35	0.39	0.45	0.43	0.31	0.49	0.57	0.64	0.55	0.55	0.44	0.41	0.38	0.26	0.15	0.30	6.70
(2)	0.09	0.10	0.12	0.12	0.08	0.13	0.15	0.17	0.15	0.15	0.12	0.11	0.10	0.07	0.05	0.08	1.78
1.5- 3.0	95	170	188	204	201	255	308	312	299	218	197	173	175	159	113	98	3165
(1)	0.88	1.57	1.74	1.89	1.86	2.36	2.85	2.88	2.76	2.02	1.82	1.60	1.62	1.47	1.04	0.91	29.26
(2)	0.23	0.42	0.46	0.50	0.49	0.63	0.76	0.77	0.74	0.54	0.48	0.43	0.43	0.39	0.28	0.24	7.78
3.1- 5.0	119	156	162	187	197	246	367	530	518	343	241	242	223	148	116	151	3946
(1)	1.10	1.44	1.50	1.73	1.82	2.27	3.39	4.90	4.79	3.17	2.23	2.24	2.06	1.37	1.07	1.40	36.49
(2)	0.29	0.38	0.40	0.46	0.48	0.60	0.90	1.30	1.27	0.84	0.59	0.59	0.55	0.36	0.29	0.37	9.70
5.1- 8.0	48	72	33	56	100	148	174	402	386	193	188	197	124	56	42	65	2284
(1)	0.44	0.67	0.31	0.52	0.92	1.37	1.61	3.72	3.57	1.78	1.74	1.82	1.15	0.52	0.39	0.60	21.12
(2)	0.12	0.18	0.08	0.14	0.25	0.36	0.43	0.99	0.95	0.47	0.46	0.48	0.30	0.14	0.10	0.16	5.61
8.1-10.4	15	10	5	2	21	26	19	56	43	32	46	51	25	9	20	14	394
(1)	0.14	0.09	0.05	0.02	0.19	0.24	0.18	0.52	0.40	0.30	0.43	0.47	0.23	0.08	0.18	0.13	3.64
(2)	0.04	0.02	0.01	0.00	0.05	0.06	0.05	0.14	0.11	0.08	0.11	0.13	0.06	0.02	0.05	0.03	0.97
OVER 10.4	4	9	9	17	24	15	20	31	36	24	24	23	13	13	4	9	275
(1)	0.04	0.08	0.08	0.16	0.22	0.14	0.18	0.29	0.33	0.22	0.22	0.21	0.12	0.12	0.04	0.08	2.54
(2)	0.01	0.02	0.02	0.04	0.06	0.04	0.05	0.08	0.09	0.06	0.06	0.06	0.03	0.03	0.01	0.02	0.68
ALL SPEEDS	319	459	446	513	576	743	950	1480	1342	870	744	730	601	413	314	369	10789
(1)	2.95	4.24	4.12	4.74	5.33	6.87	8.78	12.94	12.41	8.04	6.88	6.75	5.56	3.82	2.90	3.41	99.76
(2)	0.78	1.13	1.10	1.26	1.42	1.83	2.34	3.44	3.30	2.14	1.83	1.79	1.48	1.02	0.77	0.91	26.52

(1) = PERCENT OF ALL GOOD OB S FOR THIS PAGE

(2) = PERCENT OF ALL GOOD OB S FOR THE PERIOD

10815 HRS ON THIS PAGE 26 HRS (0.2 PCT) LESS THAN 0.3 MPS (0.1 PCT OF ALL HRS)

TABLE 2.3-20
JOINT FREQUENCY DISTRIBUTION

CLINTON POWER STATION
33 FT WIND DISTRIBUTION OF WIND DIRECTIONS AND SPEEDS 4/14/72 - 4/30/77
198-33 FT DELTA T STABILITY F - DELTA T 1.6 TO 4.0 DEG C PER 100 METERS

	DIRECTION																
SPEED (MPS)	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	N	TOTAL
0.3 - 1.4	30	50	50	42	36	49	54	59	36	44	35	44	29	25	33	39	655
(1)	0.67	1.12	1.12	0.94	0.80	1.10	1.21	1.32	0.80	0.98	0.78	0.98	0.65	0.56	0.74	0.87	14.64
(2)	0.07	0.12	0.12	0.10	0.09	0.12	0.13	0.15	0.09	0.11	0.09	0.11	0.07	0.06	0.08	0.10	1.61
1.5- 3.0	75	125	134	153	161	197	216	222	248	209	152	139	163	113	63	83	2453
(1)	1.68	2.79	3.00	3.42	3.60	4.40	4.83	4.96	5.54	4.67	3.40	3.11	3.64	2.53	1.41	1.86	54.83
(2)	0.18	0.31	0.33	0.38	0.40	0.48	0.53	0.55	0.61	0.51	0.37	0.34	0.40	0.28	0.15	0.20	6.03
3.1- 5.0	26	24	22	28	40	56	101	114	148	120	96	73	75	57	24	27	1031
(1)	0.58	0.54	0.49	0.63	0.89	1.25	2.26	2.55	3.31	2.68	2.15	1.63	1.68	1.27	0.54	0.60	23.04
(2)	0.06	0.06	0.05	0.07	0.10	0.14	0.25	0.28	0.36	0.30	0.24	0.18	0.18	0.14	0.06	0.07	2.53
5.1- 8.0	0	0	0	0	0	5	4	4	8	14	10	16	10	3	4	2	80
(1)	0.00	0.00	0.00	0.00	0.00	0.11	0.09	0.09	0.18	0.31	0.22	0.36	0.22	0.07	0.09	0.04	1.79
(2)	0.00	0.00	0.00	0.00	0.00	0.01	0.01	0.01	0.02	0.03	0.02	0.04	0.02	0.01	0.01	0.00	0.20
8.1-10.4	0	0	0	0	0	0	0	0	0	0	0	1	1	1	2	0	5
(1)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.02	0.02	0.02	0.04	0.00	0.11
(2)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01
OVER 10.4	11	21	14	22	9	13	23	18	23	17	15	12	8	5	4	9	224
(1)	0.25	0.47	0.31	0.49	0.20	0.29	0.51	0.40	0.51	0.38	0.34	0.27	0.18	0.11	0.09	0.20	5.01
(2)	0.03	0.05	0.03	0.05	0.02	0.03	0.06	0.04	0.06	0.04	0.04	0.03	0.02	0.01	0.01	0.02	0.55
ALL SPEEDS	142	220	220	245	246	320	398	417	463	404	308	285	286	204	130	160	4448
(1)	3.17	4.92	4.92	5.48	5.50	7.15	8.90	9.32	10.35	9.03	6.88	6.37	6.39	4.56	2.91	3.58	99.42
(2)	0.35	0.54	0.54	0.60	0.60	0.79	0.98	1.03	1.14	0.99	0.76	0.70	0.70	0.50	0.32	0.39	10.93

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(1) = PERCENT OF ALL GOOD OB S FOR THIS PAGE

(2) = PERCENT OF ALL GOOD OB S FOR THE PERIOD

4474 HRS ON THIS PAGE 24 HRS (0.6 PCT) LESS THAN 0.3 MPS (0.1 PCT OF ALL HRS)

TABLE 2.3-21
JOINT FREQUENCY DISTRIBUTION

CLINTON POWER STATION
33 FT WIND DISTRIBUTION OF WIND DIRECTIONS AND SPEEDS 4/14/72 - 4/30/77
198-33 FT DELTA T STABILITY G - DELTA T GREATER THAN 4.0 DEG C PER 100 METERS

	DIRECTION																
SPEED (MPS)	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	N	TOTAL
0.3 - 1.4	53	73	73	79	52	57	69	98	78	63	58	58	55	49	41	37	993
(1)	1.45	1.99	1.99	2.16	1.42	1.56	1.89	2.68	2.13	1.72	1.58	1.58	1.50	1.34	1.12	1.01	27.13
(2)	0.13	0.18	0.18	0.19	0.13	0.14	0.17	0.24	0.19	0.15	0.14	0.14	0.14	0.12	0.10	0.09	2.44
1.5- 3.0	75	138	94	93	90	160	182	189	216	151	88	94	92	96	43	57	1858
(1)	2.05	3.77	2.57	2.54	2.46	4.37	4.97	5.16	5.90	4.13	2.40	2.57	2.51	2.62	1.17	1.56	50.77
(2)	0.18	0.34	0.23	0.23	0.22	0.39	0.45	0.46	0.53	0.37	0.22	0.23	0.23	0.24	0.11	0.14	4.57
3.1- 5.0	8	9	9	10	13	19	23	23	55	28	13	17	22	27	12	7	295
(1)	0.22	0.25	0.25	0.27	0.36	0.52	0.63	0.63	1.50	0.77	0.36	0.46	0.60	0.74	0.33	0.19	8.06
(2)	0.02	0.02	0.02	0.02	0.03	0.05	0.06	0.06	0.14	0.07	0.03	0.04	0.05	0.07	0.03	0.02	0.73
5.1- 8.0	6	10	1	5	14	15	4	35	55	13	2	17	14	2	1	3	197
(1)	0.16	0.27	0.03	0.14	0.38	0.41	0.11	0.96	1.50	0.36	0.05	0.46	0.38	0.05	0.03	0.08	5.38
(2)	0.01	0.02	0.00	0.01	0.03	0.04	0.01	0.09	0.14	0.03	0.00	0.04	0.03	0.00	0.00	0.01	0.48
8.1-10.4	1	1	1	0	0	0	0	0	20	4	1	8	6	0	2	3	47
(1)	0.03	0.03	0.03	0.00	0.00	0.00	0.00	0.00	0.55	0.11	0.03	0.22	0.16	0.00	0.05	0.08	1.28
(2)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.05	0.01	0.00	0.02	0.01	0.00	0.00	0.01	0.12
OVER 10.4	8	30	27	25	15	9	16	27	16	13	16	2	5	5	2	5	221
(1)	0.22	0.82	0.74	0.68	0.41	0.25	0.44	0.74	0.44	0.36	0.44	0.05	0.14	0.14	0.05	0.14	6.04
(2)	0.02	0.07	0.07	0.06	0.04	0.02	0.04	0.07	0.04	0.03	0.04	0.00	0.01	0.01	0.00	0.01	0.54
ALL SPEEDS	151	261	205	212	184	260	294	372	440	272	178	196	194	179	101	112	3611
(1)	4.13	7.13	5.60	5.79	5.03	7.10	8.03	10.16	12.02	7.43	4.86	5.36	5.30	4.89	2.76	3.06	98.66
(2)	0.37	0.64	0.50	0.52	0.45	0.64	0.72	0.91	1.08	0.67	0.44	0.48	0.48	0.44	0.25	0.28	8.88

(1) = PERCENT OF ALL GOOD OB S FOR THIS PAGE

(2) = PERCENT OF ALL GOOD OB S FOR THE PERIOD

3660 HRS ON THIS PAGE 49 HRS (1.3 PCT) LESS THAN 0.3 MPS (0.1 PCT OF ALL HRS)

TABLE 2.3-22
JOINT FREQUENCY DISTRIBUTION

CLINTON POWER STATION
33 FT WIND DISTRIBUTION OF WIND DIRECTIONS AND SPEEDS 4/14/72 - 4/30/77
198-33 FT DELTA T ALL STABILITIES COMBINED

	DIRECTION																
SPEED (MPS)	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	N	TOTAL
0.3 - 1.4	152	212	215	209	164	195	248	290	233	235	187	195	185	137	137	146	3140
(1)	0.37	0.52	0.53	0.51	0.40	0.48	0.61	0.71	0.57	0.58	0.46	0.48	0.45	0.34	0.34	0.36	7.72
(2)	0.37	0.52	0.53	0.51	0.40	0.48	0.61	0.71	0.57	0.58	0.46	0.48	0.45	0.34	0.34	0.36	7.72
1.5- 3.0	433	690	671	692	629	839	1033	1071	1054	912	654	635	703	594	433	434	11477
(1)	1.06	1.70	1.65	1.70	1.55	2.06	2.54	2.63	2.59	2.24	1.61	1.56	1.73	1.46	1.06	1.07	28.21
(2)	1.06	1.70	1.65	1.70	1.55	2.06	2.54	2.63	2.59	2.24	1.61	1.56	1.73	1.46	1.06	1.07	28.21
3.1- 5.0	538	599	568	575	554	618	911	1240	1319	1096	803	820	909	788	572	562	12472
(1)	1.32	1.47	1.40	1.41	1.36	1.52	2.24	3.05	3.24	2.69	1.97	2.02	2.23	1.94	1.41	1.38	30.66
(2)	1.32	1.47	1.40	1.41	1.36	1.52	2.24	3.05	3.24	2.69	1.97	2.02	2.23	1.94	1.41	1.38	30.66
5.1- 8.0	377	472	229	243	314	418	503	956	1000	834	673	937	1011	668	470	420	9635
(1)	0.93	1.16	0.56	0.60	0.77	1.03	1.24	2.42	2.66	2.05	1.65	2.30	2.49	1.64	1.16	1.03	23.69
(2)	0.93	1.16	0.56	0.60	0.77	1.03	1.24	2.42	2.66	2.05	1.65	2.30	2.49	1.64	1.16	1.03	23.69
8.1-10.4	96	79	19	20	37	51	64	233	227	183	228	339	280	134	131	106	2227
(1)	0.24	0.19	0.05	0.05	0.09	0.13	0.16	0.57	0.56	0.45	0.56	0.83	0.69	0.33	0.32	0.26	5.47
(2)	0.24	0.19	0.05	0.05	0.09	0.13	0.16	0.57	0.56	0.45	0.56	0.83	0.69	0.33	0.32	0.26	5.47
OVER 10.4	51	95	65	96	81	70	88	124	150	121	177	207	135	56	40	61	1617
(1)	0.13	0.23	0.16	0.24	0.20	0.17	0.22	0.30	0.37	0.30	0.44	0.51	0.33	0.14	0.10	0.15	3.98
(2)	0.13	0.23	0.16	0.24	0.20	0.17	0.22	0.30	0.37	0.30	0.44	0.51	0.33	0.14	0.10	0.15	3.98
ALL SPEEDS	1647	2147	1767	1835	1779	2191	2847	3944	4063	3381	2722	3133	3223	2377	1783	1729	40568
(1)	4.05	5.28	4.34	4.51	4.37	5.39	7.00	9.70	9.99	8.31	6.69	7.70	7.92	5.84	4.38	4.25	99.73
(2)	4.05	5.28	4.34	4.51	4.37	5.39	7.00	9.70	9.99	8.31	6.69	7.70	7.92	5.84	4.38	4.25	99.73

(1) = PERCENT OF ALL GOOD OB S FOR THIS PAGE

(2) = PERCENT OF ALL GOOD OB S FOR THE PERIOD

40677 GOOD HRS 109 HRS (0.3 PCT) LESS THAN 0.3 MPS 44208 HRS IN THE TIME PERIOD
92.0 PCT DATA RECOVERY

TABLE 2.3-23
PREDICTED NUMBER OF HOURS OF LAKE STEAM FOG (AREA 1)

	VISIBILITY			
MONTH	100 FEET OR LESS	1/16 MILE OR LESS	3/16 MILE OR LESS	1 MILE OR LESS
January	218	299	395	419
February	120	235	349	360
March	108	151	181	195
April	25	77	151	109
May*	1	1	1	5
June*	0	0	1	4
July*	1	1	2	7
August	60	93	134	144
September	77	131	157	173
October	84	133	231	288
November	133	342	399	430
December	227	416	453	465

* Fog prediction model indicates minor steam fog for these months.
Values shown are from Peoria, Illinois (1960-1970).

1/16 mile = 330 feet
3/16 mile = 990 feet

TABLE 2.3-24
PREDICTED NUMBER OF HOURS OF LAKE STEAM FOG (AREA 2)

	VISIBILITY			
	100 FEET OR LESS	1/16 MILE OR LESS	3/16 MILE OR LESS	1 MILE OR LESS
January	108	163	163	163
February	44	148	152	152
March	41	72	76	76
April	0	33	44	46
May*	1	1	1	5
June*	0	0	1	4
July*	1	1	2	7
August	93	99	103	111
September	80	97	107	107
October	72	102	105	109
November	44	135	149	149
December	60	139	139	148

* Fog prediction model indicates minor steam fog for these months.
Values shown are from Peoria, Illinois (1960-1970).

1/16 mile = 330 feet
3/16 mile - 990 feet

TABLE 2.3-25
PREDICTED NUMBER OF HOURS OF LAKE STEAM FOG (AREA 3)

	VISIBILITY			
	100 FEET OR LESS	1/16 MILE OR LESS	3/16 MILE OR LESS	1 MILE OR LESS
January	7	65	114	144
February	3	71	99	116
March	3	39	49	68
April	0	16	20	22
May*	1	1	1	5
June*	0	0	1	4
July*	1	1	2	7
August	17	47	53	59
September	22	61	61	64
October	3	90	90	96
November	7	65	100	111
December	14	147	138	148

* Fog prediction model indicates minor steam fog for these months.
Values shown are from Peoria, Illinois (1960-1970).

1/16 mile = 330 feet
3/16 mile = 990 feet

TABLE 2.3-26
PREDICTED NUMBER OF HOURS OF LAKE STEAM FOG (AREA 4)

		VISIBILITY		
	100 FEET OR LESS	1/16 MILE OR LESS	3/16 MILE OR LESS	1 MILE OR LESS
January	7	7	17	50
February	3	3	5	34
March	3	3	4	28
April	0	0	1	8
May*	1	1	1	5
June*	0	0	1	4
July*	1	1	2	7
August	0	0	1	6
September	1	7	8	14
October	3	10	14	21
November	4	4	9	26
December	2	2	6	24

* Fog prediction model indicates minor steam fog for these months.
Values shown are from Peoria, Illinois (1960-1979).

1/16 mile = 330 feet
3/16 mile = 990 feet

TABLE 2.3-27
PREDICTED NUMBER OF HOURS OF LAKE STEAM FOG (AREA 5)

		VISIBILITY		
MONTH	100 FEET OR LESS	1/16 MILE OR LESS	3/16 MILE OR LESS	1 MILE OR LESS
January	7	20	38	87
February	3	10	39	75
March	3	5	20	38
April	0	0	1	6
May*	1	1	1	5
June*	0	0	1	4
July*	1	1	2	7
August	0	17	24	47
September	1	29	48	63
October	3	14	46	59
November	4	26	34	48
December	2	7	21	60

* Fog prediction model indicates minor steam fcg for these months.
Values shown are from Peoria, Illinois (1960-1970).

1/16 mile = 330 feet
3/16 mile = 990 feet

TABLE 2.3-28
PREDICTED NUMBER OF HOURS OF LAKE STEAM FOG (AREA 6)

MONTH	100 FEET OR LESS	VISIBILITY 1/16 MILE OR LESS	3/16 MILE OR LESS	1 MILE OR LESS
January	7	87	123	156
February	3	21	47	88
March	3	8	17	38
April	0	6	11	18
May*	1	1	1	5
June*	0	0	1	4
July*	1	1	2	7
August	3	12	24	37
September	9	57	82	88
October	6	81	141	147
November	15	74	125	168
December	6	60	102	144

* Fog prediction model indicates minor steam fog for these months.
Values shown are from Peoria, Illinois (1960-1970).

1/16 mile = 330 feet
3/16 mile = 990 feet

TABLE 2.3-29
PREDICTED NUMBER OF HOURS OF LAKE STEAM FOG (AREA 7)

MONTH	100 FEET OR LESS	VISIBILITY 1/16 MILE OR LESS	VISIBILITY 3/16 MILE OR LESS	1 MILE OR LESS
January	3	16	61	128
February	1	15	58	98
March	0	3	4	40
April	0	0	22	45
May*	1	1	1	5
June*	0	0	1	4
July*	1	1	2	7
August	0	20	22	50
September	4	23	33	52
October	4	22	28	39
November	2	7	23	79
December	1	16	61	138

* Fog prediction model indicates minor steam fog for these months.
Values shown are from Peoria, Illinois (1960-1970).

1/16 mile = 330 feet
3/16 mile = 990 feet

TABLE 2.3-30
CLINTON POWER STATION SITE ACCIDENT χ/Q CALCULATIONS

1 HOUR AVERAGING PERIOD

DOWNWIND SECTION	DISTANCE (METERS)	(SEC. PER CUBIC METER) 5 PCT χ/Q	50 PCT χ/Q
SSW	975	0.182E-03	0.274E-04
SW	975	0.190E-03	0.294E-04
WSW	975	0.210E-03	0.349E-04
W	975	0.211E-03	0.376E-04
WNW	975	0.169E-03	0.361E-04
NW	975	0.177E-03	0.377E-04
NNW	975	0.168E-03	0.350E-04
N	975	0.163E-03	0.291E-04
NNE	975	0.151E-03	0.311E-04
NE	975	0.154E-03	0.289E-04
ENE	975	0.153E-03	0.279E-04
E	975	0.150E-03	0.254E-04
ESE	975	0.143E-03	0.248E-04
SE	975	0.149E-03	0.258E-04
SSE	975	0.164E-03	0.254E-04
S	975	0.156E-03	0.277E-04
All Direction Case		0.178E-03	0.305E-04

Period of Record - May 1972-April 1977.

TABLE 2.3-31
CLINTON POWER STATION SITE ACCIDENT χ/Q CALCULATIONS

1 HOUR AVERAGING PERIOD

DOWNWIND SECTOR	DISTANCE (METERS)	(SEC. PER CUBIC METER) 5 PCT χ/Q	50 PCT χ/Q
SSW	4018	0.427E-04	0.347E-05
SW	4018	0.449E-04	0.379E-05
WSW	4018	0.475E-04	0.488E-05
W	4018	0.476E-04	0.528E-05
WNW	4018	0.379E-04	0.505E-05
NW	4018	0.401E-04	0.527E-05
NNW	4018	0.379E-04	0.473E-05
N	4018	0.342E-04	0.377E-05
NNE	4018	0.336E-04	0.425E-05
NE	4018	0.344E-04	0.374E-05
ENE	4018	0.354E-04	0.363E-05
E	4018	0.310E-04	0.315E-05
ESE	4018	0.282E-04	0.303E-05
SE	4018	0.331E-04	0.313E-05
SSE	4018	0.372E-04	0.304E-05
S	4018	0.367E-04	0.353E-05
All Direction Case		0.415E-04	0.426E-05

Period of Record - May 1972-April 1977.

TABLE 2.3-32
CLINTON POWER STATION SITE ACCIDENT χ/Q CALCULATIONS

1 HOUR AVERAGING PERIOD

DOWNWIND SECTOR	DISTANCE (METERS)	(SEC. PER CUBIC METER) 5 PCT χ/Q	50 PCT χ/Q
SSW	4727	0.352E-04	0.271E-05
SW	5121	0.343E-04	0.270E-05
WSW	3482	0.568E-04	0.592E-05
W	2377	0.878E-04	0.110E-04
WNW	1508	0.115E-03	0.203E-04
NW	1585	0.114E-03	0.197E-04
NNW	1615	0.108E-03	0.174E-04
N	1585	0.105E-03	0.144E-04
NNE	1615	0.944E-04	0.156E-04
NE	1402	0.112E-03	0.174E-04
ENE	1189	0.127E-03	0.210E-04
E	1158	0.128E-03	0.197E-04
ESE	4724	0.232E-04	0.239E-05
SE	4077	0.328E-04	0.305E-05
SSE	3353	0.467E-04	0.395E-05
S	3353	0.453E-04	0.455E-05

Period of Record - May 1972-April 1977.

TABLE 2.3-33
CLINTON POWER STATION SITE ACCIDENT χ/Q CALCULATIONS

2 HOUR AVERAGING PERIOD

DOWNWIND SECTOR	DISTANCE (METERS)	(SEC. PER CUBIC METER) 5 PCT χ/Q	50 PCT χ/Q
SSW	975	0.120E-03	0.193E-04
SW	975	0.137E-03	0.223E-04
WSW	975	0.141E-03	0.247E-04
W	975	0.141E-03	0.251E-04
WNW	975	0.118E-03	0.247E-04
NW	975	0.137E-03	0.247E-04
NNW	975	0.131E-03	0.241E-04
N	975	0.124E-03	0.214E-04
NNE	975	0.115E-03	0.226E-04
NE	975	0.113E-03	0.198E-04
ENE	975	0.101E-03	0.197E-04
E	975	0.982E-04	0.181E-04
ESE	975	0.945E-04	0.177E-04
SE	975	0.102E-03	0.173E-04
SSE	975	0.107E-03	0.169E-04
S	975	0.112E-03	0.200E-04
All Direction Case		0.126E-03	0.231E-04

Period of Record - May 1972-April 1977.

TABLE 2.3-34
CLINTON POWER STATION SITE ACCIDENT χ/Q CALCULATIONS

2 HOUR AVERAGING PERIOD

DOWNWIND SECTOR	DISTANCE (METERS)	(SEC. PER CUBIC METER) 5 PCT χ/Q	50 PCT χ/Q
SSW	4018	0.284E-04	0.256E-05
SW	4018	0.315E-04	0.287E-05
WSW	4018	0.317E-04	0.346E-05
W	4018	0.305E-04	0.366E-05
WNW	4018	0.248E-04	0.356E-05
NW	4018	0.294E-04	0.357E-05
NNW	4018	0.266E-04	0.331E-05
N	4018	0.247E-04	0.279E-05
NNE	4018	0.246E-04	0.299E-05
NE	4018	0.247E-04	0.261E-05
ENE	4018	0.230E-04	0.264E-05
E	4018	0.217E-04	0.236E-05
ESE	4018	0.194E-04	0.229E-05
SE	4018	0.217E-04	0.220E-05
SSE	4018	0.234E-04	0.216E-05
S	4018	0.237E-04	0.264E-05
All Direction Case		0.272E-04	0.308E-05

Period of Record - May 1972-April 1977.

TABLE 2.3-35

CLINTON POWER STATION SITE ACCIDENT χ/Q CALCULATIONS

2 HOUR AVERAGING PERIOD

DOWNWIND SECTOR	DISTANCE (METERS)	(SEC. PER CUBIC METER) 5 PCT χ/Q	50 PCT χ/Q
SSW	4727	0.230E-04	0.203E-05
SW	5121	0.238E-04	0.206E-05
WSW	3482	0.388E-04	0.428E-05
W	2377	0.575E-04	0.768E-05
WNW	1508	0.774E-04	0.143E-04
NW	1585	0.862E-04	0.132E-04
NNW	1615	0.807E-04	0.124E-04
N	1585	0.763E-04	0.107E-04
NNE	1615	0.697E-04	0.112E-04
NE	1402	0.772E-04	0.119E-04
ENE	1189	0.813E-04	0.153E-04
E	1158	0.814E-04	0.142E-04
ESE	4724	0.157E-04	0.180E-05
SE	4077	0.214E-04	0.216E-05
SSE	3353	0.283E-04	0.283E-05
S	3353	0.296E-04	0.343E-05

Period of Record - May 1972-April 1977.

TABLE 2.3-36
CLINTON POWER STATION SITE ACCIDENT χ/Q CALCULATIONS

8 HOUR AVERAGING PERIOD

DOWNWIND SECTOR	DISTANCE (METERS)	(SEC. PER CUBIC METER) 5 PCT χ/Q	50 PCT χ/Q
SSW	975	0.517E-04	0.891E-05
SW	975	0.660E-04	0.104E-04
WSW	975	0.606E-04	0.113E-04
W	975	0.647E-04	0.124E-04
WNW	975	0.529E-04	0.111E-04
NW	975	0.605E-04	0.111E-04
NNW	975	0.621E-04	0.111E-04
N	975	0.596E-04	0.108E-04
NNE	975	0.605E-04	0.102E-04
NE	975	0.548E-04	0.890E-05
ENE	975	0.489E-04	0.804E-05
E	975	0.464E-04	0.833E-05
ESE	975	0.490E-04	0.887E-05
SE	975	0.450E-04	0.836E-05
SSE	975	0.431E-04	0.734E-05
S	975	0.488E-04	0.890E-05
All Direction Case		0.600E-04	0.104E-04

Period of Record - May 1972-April 1977.

TABLE 2.3-37

CLINTON POWER STATION SITE ACCIDENT χ/Q CALCULATIONS

8 HOUR AVERAGING PERIOD

DOWNWIND SECTOR	DISTANCE (METERS	(SEC. PER CUBIC METER) 5 PCT χ/Q	50 PCT χ/Q
SSW	4018	0.118E-04	0.123E-05
SW	4018	0.142E-04	0.147E-05
WSW	4018	0.129E-04	0.162E-05
W	4018	0.134E-04	0.179E-05
WNW	4018	0.104E-04	0.162E-05
NW	4018	0.125E-04	0.160E-05
NNW	4018	0.124E-04	0.155E-05
N	4018	0.118E-04	0.147E-05
NNE	4018	0.117E-04	0.139E-05
NE	4018	0.112E-04	0.121E-05
ENE	4018	0.964E-05	0.113E-05
E	4018	0.946E-05	0.115E-05
ESE	4018	0.100E-04	0.118E-05
SE	4018	0.931E-05	0.114E-05
SSE	4018	0.943E-05	0.101E-05
S	4018	0.921E-05	0.123E-05
All Direction Case		0.125E-04	0.147E-05

Period of Record - May 1972-April 1977.

TABLE 2.3-38
CLINTON POWER STATION SITE ACCIDENT χ/Q CALCULATIONS

16 HOUR AVERAGING PERIOD

DOWNWIND SECTOR	DISTANCE (METERS)	(SEC. PER CUBIC METER) 5 PCT χ/Q	50 PCT χ/Q
SSW	975	0.327E-04	0.588E-05
SW	975	0.403E-04	0.719E-05
WSW	975	0.396E-04	0.714E-05
W	975	0.434E-04	0.859E-05
WNW	975	0.332E-04	0.727E-05
NW	975	0.393E-04	0.725E-05
NNW	975	0.406E-04	0.753E-05
N	975	0.407E-04	0.771E-05
NNE	975	0.403E-04	0.693E-05
NE	975	0.380E-04	0.580E-05
ENE	975	0.320E-04	0.513E-05
E	975	0.312E-04	0.565E-05
ESE	975	0.342E-04	0.602E-05
SE	975	0.307E-04	0.537E-05
SSE	975	0.289E-04	0.469E-05
S	975	0.290E-04	0.584E-05
All Direction Case		0.403E-04	0.710E-05

Period of Record - May 1972-April 1977.

TABLE 2.3-39

CLINTON POWER STATION SITE ACCIDENT χ/Q CALCULATIONS

16 HOUR AVERAGING PERIOD

DOWNWIND SECTOR	DISTANCE (METERS)	(SEC. PER CUBIC METER) 5 PCT χ/Q	50 PCT χ/Q
SSW	4018	0.712E-05	0.860E-06
SW	4018	0.869E-05	0.107E-05
WSW	4018	0.824E-05	0.105E-05
W	4018	0.905E-05	0.131E-05
WNW	4018	0.669E-05	0.112E-05
NW	4018	0.775E-05	0.109E-05
NNW	4018	0.764E-05	0.113E-05
N	4018	0.797E-05	0.111E-05
NNE	4018	0.770E-05	0.997E-06
NE	4018	0.758E-05	0.815E-06
ENE	4018	0.647E-05	0.736E-06
E	4018	0.661E-05	0.792E-06
ESE	4018	0.673E-05	0.841E-06
SE	4018	0.610E-05	0.740E-06
SSE	4018	0.596E-05	0.633E-06
S	4018	0.579E-05	0.810E-06
All Direction Case		0.820E-05	0.100E-05

Period of Record - May 1972-April 1977.

TABLE 2.3-40
CLINTON POWER STATION SITE ACCIDENT χ/Q CALCULATIONS

72 HOUR AVERAGING PERIOD

DOWNWIND SECTOR	DISTANCE (METERS)	(SEC. PER CUBIC METER) 5 PCT χ/Q	50 PCT χ/Q
SSW	975	0.125E-04	0.228E-05
SW	975	0.174E-04	0.318E-05
WSW	975	0.148E-04	0.303E-05
W	975	0.162E-04	0.350E-05
WNW	975	0.132E-04	0.305E-05
NW	975	0.151E-04	0.312E-05
NNW	975	0.181E-04	0.358E-05
N	975	0.185E-04	0.399E-05
NNE	975	0.182E-04	0.370E-05
NE	975	0.157E-04	0.307E-05
ENE	975	0.135E-04	0.244E-05
E	975	0.128E-04	0.269E-05
ESE	975	0.144E-04	0.269E-05
SE	975	0.136E-04	0.228E-05
SSE	975	0.123E-04	0.191E-05
S	975	0.130E-04	0.204E-05
All Direction Case		0.171E-04	0.320E-05

Period of Record - May 1972-April 1977.

TABLE 2.3-41
CLINTON POWER STATION SITE ACCIDENT χ/Q CALCULATIONS

72 HOUR AVERAGING PERIOD

DOWNWIND SECTOR	DISTANCE (METERS)	(SEC. PER CUBIC METER) 5 PCT χ/Q	50 PCT χ/Q
SSW	4018	0.258E-05	0.360E-06
SW	4018	0.348E-05	0.478E-06
WSW	4018	0.317E-05	0.489E-06
W	4018	0.354E-05	0.551E-06
WNW	4018	0.248E-05	0.487E-06
NW	4018	0.292E-05	0.521E-06
NNW	4018	0.356E-05	0.541E-06
N	4018	0.343E-05	0.600E-06
NNE	4018	0.335E-05	0.575E-06
NE	4018	0.329E-05	0.457E-06
ENE	4018	0.268E-05	0.392E-06
E	4018	0.254E-05	0.391E-06
ESE	4018	0.277E-05	0.390E-06
SE	4018	0.262E-05	0.327E-06
SSE	4018	0.239E-05	0.267E-06
S	4018	0.246E-05	0.317E-06
All Direction Case		0.330E-05	0.490E-06

Period of Record - May 1972-April 1977.

TABLE 2.3-42
CLINTON POWER STATION SITE ACCIDENT χ/Q CALCULATIONS

624 HOUR AVERAGING PERIOD

DOWNWIND SECTOR	DISTANCE (METERS)	(SEC. PER CUBIC METER) 5 PCT χ/Q	50 PCT χ/Q
SSW	975	0.488E-05	0.159E-05
SW	975	0.670E-05	0.229E-05
WSW	975	0.643E-05	0.244E-05
W	975	0.711E-05	0.258E-05
WNW	975	0.584E-05	0.235E-05
NW	975	0.746E-05	0.312E-05
NNW	975	0.888E-05	0.322E-05
N	975	0.984E-05	0.402E-05
NNE	975	0.886E-05	0.401E-05
NE	975	0.750E-05	0.351E-05
ENE	975	0.706E-05	0.229E-05
E	975	0.654E-05	0.287E-05
ESE	975	0.826E-05	0.275E-05
SE	975	0.568E-05	0.215E-05
SSE	975	0.493E-05	0.152E-05
S	975	0.551E-05	0.153E-05
All Direction Case		0.810E-05	0.296E-05

Period of Record - May 1972-April 1977.

TABLE 2.3-43

CLINTON POWER STATION SITE ACCIDENT χ/Q CALCULATIONS

624 HOUR AVERAGING PERIOD

DOWNWIND SECTOR	DISTANCE (METERS)	(SEC. PER CUBIC METER) 5 PCT χ/Q	50 PCT χ/Q
SSW	4018	0.101E-05	0.270E-06
SW	4018	0.138E-05	0.382E-06
WSW	4018	0.120E-05	0.402E-06
W	4018	0.149E-05	0.435E-06
WNW	4018	0.114E-05	0.391E-06
NW	4018	0.145E-05	0.533E-06
NNW	4018	0.167E-05	0.552E-06
N	4018	0.178E-05	0.661E-06
NNE	4018	0.155E-05	0.664E-06
NE	4018	0.149E-05	0.605E-06
ENE	4018	0.139E-05	0.386E-06
E	4018	0.122E-05	0.491E-06
ESE	4018	0.153E-05	0.422E-06
SE	4018	0.104E-05	0.333E-06
SSE	4018	0.926E-06	0.231E-06
S	4018	0.103E-05	0.246E-06
All Direction Case		0.155E-05	0.480E-06

Period of Record - May 1972-April 1977.

TABLE 2.3-44
ANNUAL AVERAGE χ/Q

CLINTON: 5/72 THROUGH 4/77
MEAN ANNUAL χ/Q BY SECTOR; GROUND-LEVEL RELEASE
SECTOR AVERAGE

DOWNWIND SECTOR	DOWNWIND DISTANCE (miles) 0.5	1.5	2.5	3.5
SSW	8.30E-06	1.20E-06	5.00E-07	2.70E-07
SW	1.00E-05	1.50E-06	6.30E-07	3.40E-07
WSW	1.10E-05	1.60E-06	6.40E-07	3.50E-07
W	8.40E-06	1.40E-06	6.00E-07	3.40E-07
WNW	9.30E-06	1.50E-06	6.20E-07	3.50E-07
NW	9.10E-06	1.50E-06	6.30E-07	3.70E-07
NNW	1.00E-05	1.70E-06	7.20E-07	4.10E-07
N	1.48E-05	2.30E-06	1.03E-06	5.70E-07
NNE	1.80E-05	3.00E-06	1.30E-06	7.00E-07
NE	1.60E-05	2.50E-06	1.00E-06	5.60E-07
ENE	1.00E-05	1.60E-06	6.80E-07	3.90E-07
E	9.00E-06	1.40E-06	5.70E-07	3.30E-07
ESE	6.80E-06	1.20E-06	5.20E-07	3.00E-07
SE	8.30E-06	1.30E-06	5.20E-07	2.90E-07
SSE	7.40E-06	1.20E-06	5.30E-07	2.90E-07
S	6.20E-06	1.00E-06	4.10E-07	2.30E-07
ALL	1.63E-04	2.59E-05	1.09E-05	6.09E-06

44,232 HRS EXAMINED

DOWNWIND SECTOR	4.5	5.0	7.5	10.0
SSW	1.80E-07	1.40E-07	7.10E-08	4.20E-08
SW	2.20E-07	1.80E-07	8.80E-08	5.40E-08
WSW	2.50E-07	1.90E-07	9.20E-08	5.60E-08
W	2.30E-07	1.90E-07	9.90E-08	6.20E-08
WNW	2.30E-07	1.90E-07	9.50E-08	5.80E-08
NW	2.40E-07	2.00E-07	1.00E-07	6.50E-08
NNW	2.70E-07	2.20E-07	1.10E-07	6.90E-08
N	3.80E-07	3.20E-07	1.65E-07	1.04E-07
NNE	4.70E-07	4.10E-07	2.10E-07	1.30E-07
NE	3.70E-07	3.10E-07	1.50E-07	9.30E-08

TABLE 2.3-44 (CONT'D)
ANNUAL AVERAGE χ/Q

CLINTON: 5/72 THROUGH 4/77
MEAN ANNUAL χ/Q BY SECTOR; GROUND-LEVEL RELEASE
SECTOR AVERAGE

DOWNWIND DISTANCE (miles)

ENE	2.50E-07	2.10E-07	1.00E-07	6.80E-08
E	2.20E-07	1.70E-07	8.80E-08	5.30E-08
ESE	1.90E-07	1.70E-07	8.70E-08	5.50E-08
SE	1.90E-07	1.60E-07	7.80E-08	4.70E-08
SSE	1.90E-07	1.60E-07	8.30E-08	5.00E-08
S	1.50E-07	1.30E-07	6.50E-08	4.00E-08
ALL	4.03E-06	3.35E-06	1.68E-06	1.05E-06

44,232 HRS EXAMINED

DOWNWIND SECTOR	15.0	25.0	35.0	45.0
SSW	2.10E-08	8.50E-09	4.70E-09	3.00E-09
SW	2.60E-08	1.00E-08	5.70E-09	3.70E-09
WSW	2.70E-08	1.10E-08	6.00E-09	3.90E-09
W	3.10E-08	1.30E-08	7.60E-09	5.00E-09
WNW	2.90E-08	1.20E-08	7.20E-09	4.60E-09
NW	3.30E-08	1.40E-08	8.00E-09	5.40E-09
NNW	3.50E-08	1.50E-08	8.50E-09	5.60E-09
N	5.20E-08	2.30E-08	1.30E-08	8.50E-09
NNE	6.50E-08	2.80E-08	1.60E-08	1.00E-08
NE	4.60E-08	1.90E-08	1.10E-08	6.90E-09
ENE	3.30E-08	1.40E-08	8.20E-09	5.50E-09
E	2.70E-08	1.10E-08	6.30E-09	4.10E-09
ESE	2.80E-08	1.20E-08	7.30E-09	4.70E-09
SE	2.30E-08	9.60E-09	5.20E-09	3.50E-09
SSE	2.60E-08	1.10E-08	6.30E-09	4.00E-09
S	2.00E-08	8.50E-09	4.90E-09	3.20E-09
ALL	5.22E-07	2.20E-07	1.26E-07	8.16E-08

44,232 HRS EXAMINED

TABLE 2.3-45
CLINTON POWER STATION JOINT WIND-STABILITY CLASS OCCURRENCE
FREQUENCY DISTRIBUTION (2000-2002)
10 METER TOWER LEVEL

		Wind Direction Category																
	Wind Speed Category(1)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	Total
1 (A)	2	0	1	2	6	4	16	13	19	6	5	7	5	3	2	2	1	92
	3	18	7	62	63	76	89	94	105	134	89	99	41	41	45	40	25	1028
	4	41	37	97	33	25	14	50	32	140	132	129	71	89	80	91	43	1104
	5	19	14	23	5	4	0	9	26	71	65	64	42	78	77	53	22	572
	6	0	0	1	0	0	0	1	2	9	5	2	5	19	18	9	5	76
	7	0	0	0	0	0	0	0	1	0	0	0	2	3	0	2	0	8
2 (B)	2	4	0	5	6	7	13	15	7	16	9	9	6	3	4	1	0	105
	3	21	22	48	32	24	20	43	39	54	54	51	32	36	39	41	21	577
	4	42	33	30	19	10	1	13	33	50	63	86	60	54	54	59	29	636
	5	13	8	12	2	3	1	4	19	32	42	24	34	29	25	16	7	271
	6	1	1	2	0	0	0	0	2	8	3	3	4	9	6	4	3	46
	7	0	0	0	0	0	0	0	0	1	0	0	1	4	0	1	0	7
3 (C)	2	2	3	6	5	6	9	10	10	9	5	10	6	1	8	8	3	101
	3	30	18	51	35	23	23	37	32	36	27	40	29	32	37	44	40	534
	4	41	41	29	18	6	8	19	33	64	50	58	45	49	54	42	29	586
	5	17	21	22	0	1	0	8	12	25	20	19	27	31	52	33	13	301
	6	0	9	5	0	0	0	0	1	6	4	2	16	9	12	7	1	72
	7	0	0	0	0	0	0	0	0	2	0	0	1	1	1	0	0	5
4 (D)	2	19	23	32	38	53	70	35	38	22	33	42	22	32	23	24	21	527
	3	153	160	231	183	157	180	210	230	173	181	161	102	170	159	177	146	2773
	4	232	219	226	106	51	72	147	275	365	360	183	172	312	315	296	187	3518
	5	60	79	73	7	1	2	27	71	183	210	60	93	215	193	130	53	1457
	6	2	9	8	0	0	0	0	5	47	30	9	24	42	31	10	4	221
	7	1	1	1	0	0	0	0	0	0	0	1	1	9	4	4	1	23
5 (E)	2	18	36	67	65	87	109	89	67	63	68	55	55	37	31	30	17	894
	3	77	96	197	148	142	152	214	361	407	314	198	177	154	161	135	102	3035
	4	42	43	33	21	11	13	67	178	350	347	160	108	124	93	38	46	1674
	5	0	6	1	0	0	0	5	17	112	76	33	24	42	10	10	5	341
	6	0	0	0	0	0	0	0	3	18	6	1	1	1	2	1	0	33
	7	0	0	3	0	0	0	0	1	0	0	0	0	0	0	0	0	4
6 (F)	2	21	50	90	63	60	64	51	48	52	63	44	53	46	40	30	16	791
	3	38	83	136	72	37	21	89	98	102	111	99	84	73	51	78	23	1195
	4	1	8	10	16	11	1	3	11	23	24	17	35	9	15	14	6	204
	5	1	0	0	1	0	0	0	0	0	0	4	13	2	0	1	3	25
	6	0	0	0	0	0	0	0	0	0	0	0	3	0	0	0	0	3
	7	0	1	3	0	0	0	0	0	1	0	0	0	0	0	0	0	5
7 (G)	2	20	74	127	50	45	35	27	25	21	24	30	39	38	50	42	16	663
	3	10	65	102	21	17	4	21	15	21	19	31	28	16	25	53	4	452
	4	0	1	1	5	5	0	0	0	0	0	4	4	0	0	7	1	28
	5	0	0	0	2	4	0	0	0	0	0	4	4	0	0	0	0	14
	6	0	0	0	0	0	0	0	0	0	0	0	1	0	0	0	0	1
	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	>=0.5 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. Calm occurrences by stability class: A=0, B=2, C=1, D=3, E=7, F=10, G=10

TABLE 2.3-46
CLINTON POWER STATION JOINT WIND-STABILITY CLASS OCCURRENCE
FREQUENCY DISTRIBUTION (2000-2002)
60 METER TOWER LEVEL

		Wind Direction Category																
	Wind Speed Category(1)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	Total
1 (A)	2	0	1	2	2	2	20	20	15	7	5	3	1	2	1	1	2	84
	3	8	10	28	31	50	65	73	76	63	54	53	35	25	19	18	14	622
	4	29	43	47	49	54	23	47	69	95	128	112	51	57	46	79	47	976
	5	36	17	71	22	19	4	23	27	85	112	84	58	90	47	57	37	789
	6	1	5	18	6	5	0	3	22	46	35	26	11	33	36	35	17	299
	7	0	9	7	2	2	0	2	15	12	6	7	7	23	14	8	9	123
2 (B)	2	2	0	0	2	7	9	16	15	6	13	7	5	3	3	2	1	91
	3	11	20	18	18	17	28	23	19	31	32	34	19	21	26	23	17	357
	4	35	22	29	15	13	10	16	33	39	49	75	50	49	29	30	53	547
	5	27	14	18	13	10	0	8	20	42	65	50	38	39	30	32	16	422
	6	2	4	15	3	1	0	1	14	17	24	9	13	14	14	11	8	150
	7	4	5	6	0	1	2	1	7	13	4	2	9	15	6	4	3	82
3 (C)	2	0	3	2	2	5	5	10	5	1	5	6	3	3	6	8	3	67
	3	21	17	20	15	17	23	20	28	22	16	26	19	21	14	31	21	331
	4	51	24	21	9	16	9	21	28	30	41	52	36	42	24	40	35	479
	5	33	22	16	13	4	2	14	20	42	46	44	23	29	40	27	32	407
	6	5	17	12	7	2	0	4	9	16	12	14	20	26	18	26	10	198
	7	2	10	17	0	1	0	3	4	11	9	1	19	15	8	15	7	122
4 (D)	2	11	11	15	8	16	21	16	12	11	13	10	12	18	9	13	10	206
	3	94	69	58	38	63	107	70	66	56	79	70	44	68	69	92	89	1132
	4	215	129	161	98	104	86	168	175	195	177	159	114	158	140	187	191	2457
	5	199	183	158	129	80	45	73	204	327	314	168	165	224	229	284	165	2947
	6	38	62	104	41	24	3	34	105	184	160	55	80	174	100	114	39	1317
	7	11	18	38	9	2	4	11	36	98	66	14	37	74	21	33	14	486
5 (E)	2	12	9	11	3	5	11	10	6	12	5	8	13	7	4	6	5	127
	3	36	24	27	27	39	112	96	72	45	62	47	42	35	27	37	30	758
	4	80	78	90	106	92	86	158	257	253	187	159	140	113	88	110	99	2096
	5	70	91	107	73	82	15	65	244	422	380	236	146	130	96	85	51	2293
	6	1	11	10	5	17	1	17	59	149	138	63	31	25	9	6	6	548
	7	9	10	4	1	2	8	2	10	73	15	7	9	10	2	1	2	165
6 (F)	2	7	6	2	0	13	12	0	5	9	4	6	4	5	5	7	3	88
	3	26	8	21	21	25	46	35	26	31	28	35	14	12	19	14	15	376
	4	38	33	52	55	50	29	43	58	91	69	59	61	46	45	36	60	825
	5	18	72	102	72	49	2	13	31	83	83	111	87	45	20	29	27	844
	6	0	2	8	3	4	0	2	6	9	13	9	10	2	0	0	4	72
	7	6	8	1	0	0	0	0	0	1	0	0	5	0	1	2	1	25
7 (G)	2	6	2	7	7	3	16	3	1	9	8	7	5	4	3	6	8	95
	3	5	15	12	10	25	32	17	22	13	14	15	12	13	14	16	20	255
	4	15	25	36	56	58	19	14	23	28	42	27	29	20	13	39	25	469
	5	10	25	57	52	36	0	1	6	13	16	29	30	9	3	10	18	315
	6	0	2	2	4	7	0	0	0	0	0	2	6	0	0	0	1	24
	7	1	0	1	0	0	0	0	1	0	0	1	4	0	0	0	0	8

Notes:

1) Wind speed categories defined as follows:

Category	Wind Speed (mph)
2	>=0.5 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. Calm occurrences by stability class: A=0, B=2, C=1, D=3, E=7, F=10, G=10

ATTACHMENT A2.3

ANALYTICAL FOG MODEL

ATTACHMENT A2.3 - ANALYTICAL FOG MODEL

The basic problem of predicting steam fog from a warm lake requires the following calculations:

a. Determine the evaporation per unit area of the lake.

b. Estimate the amount of evaporated water vapor that will be condensed due to existing ambient conditions.

c. Calculate the expected downwind concentrations of condensed water vapor.

d. Relate the calculated condensed water vapor to horizontal visibility.

A model has been developed by TRC that calculates values for a, b, and c. A relationship between visibility and condensed water vapor has been established and is used to relate the computed values to expected visibility.

The model has been calibrated by means of conditions recorded at a large cooling pond for a nuclear power station (Reference 1). The observed data included water temperatures at various parts of the lake, ambient air temperature and relative humidity, and observations of ambient fog and lake steaming by trained weather observers.

For the Clinton Power Station (CPS) study, only air temperatures above –4x F were used. This allows most of the condensed water vapor to form water droplets rather than condense directly into the solid phase as ice crystals.

Evaporation from a unit of surface on the lake per unit of time in a layer from h_1 to h_2 is computed by Equation A2.3-1:

$$E = \frac{\kappa 2\rho \ (q_1 - q_2)\ (u_2 - u_1)}{1n(h_2 / h_1)^2} \qquad \text{(cgs units)} \qquad \text{(A2.3-1)}$$

Where k is the Von Karmen coefficient: r is the density of air; h2 and h1 are the heights of the top and bottom, respectively, of the layer in which evaporation takes place; q_1 and q_2 are the specific humidities: and u_1 and u_2 are the wind speeds (References 2 and 3). The value of E from this equation is converted to an equivalent line source value using the dimensions of the unit area. The proper values of wind fetch for use in defining a unit area for conversion into a line source were examined and evaluated in the calibration and verification of the model. The standard line source diffusion equation for surface concentration is as follows (Reference 4):

$$X(x,y,o) = \frac{E}{\sqrt{2\pi} \quad \mu\sigma_Z} \quad \text{erf} \ \frac{(y + y_o)}{(\sqrt{2} \ \sigma_y)} - \text{erf} \ \frac{(y - y_o)}{(\sqrt{2}\,\sigma_y)} \qquad \text{(A2.3-2)}$$

The equation is used to calculate concentrations of water vapor and condensed water vapor for an orthogonal array of points downwind. In Equation A2.3-2, E is the line source strength determined from Equation A2.3-1, erf is the error function, y_o is the half-width of the source, and σ_y and σ_z are the horizontal and vertical diffusion parameters, respectively. The predicted concentrations are used to determine the amount of condensed water vapor that would exist at the downwind points, after allowing the ambient air to reach saturation.

The downwind concentrations are related to horizontal visibility by the following relationship:

$$w = c \frac{k\ r}{V} \qquad \text{(A2.3-3)}$$

where w is the liquid water content of the air and V is the resulting visibility. The term c is an empirical constant and k is a factor that accounts for drop size distribution around the average radius r. This equation was derived in other research studies on fog where both liquid water content and the drop size distribution were measured. However, such studies on natural fog (Reference 5) include a predominate number of warm fog cases in which the drop size distribution is different than for cold fogs. Therefore, the data for natural fog are used when the ambient air temperature is 36° F or higher. For cold fogs, a mean drop size radius of 10m was used with a factor of k = 1.2 in Equation A2.3-1 (Reference 1). This produces a curve that is used when the air temperature is 28° F or less, and is in good agreement with the results of a U.S. Army study on arctic fogs (Reference 6). A log-log plot of Equation A2.3-3 is presented in Figure A2.3-1 for the warm fog and cold fog cases. An interpolation is used between the two curves for transition temperatures between 28° F and 36° F.

Occurrences of overpredicting downwind concentrations of water vapor were investigated as part of the model development. The problem was related to the evaporative processes on a parcel of air as it travels across the lake. That is, the term (q_1 - q_2) from Equation A2.3-1 decreases with travel time because of the following dynamic effects;

a. The specific humidity of the air q_2, initially is a function of the dew point, and is normally less than the saturation specific humidity. As the air receives water vapor from the pond, saturation is reached, increasing the value of q_2.

b. As further moisture is received by the air after it has reached saturation, the water vapor condenses into liquid water, releasing the latent heat of condensation of the water vapor. This further increases q_2.

c. As fog is formed, heat radiated from the pond is reflected and absorbed by the water droplets, further increasing the air temperature and hence, q_2.

d. Convection of heat from the pond surface to the atmosphere still further increases q_2.

As the value of q_2 increases by the previous methods, the term (q_1 - q_2) decreases and hence the evaporation into a parcel of air decreases as it travels across the lake. The first two mechanisms are quantifiable and were used to determine the weighting factor for adjusting the evaporation rate with travel time. Radiation and convective effects were not computed and thus were empirically accounted for in the calibration of the model.

Use of the Model

Predicted water temperatures for six areas of the lake evaluated to date are reduced to representative (monthly) values. The lake is divided into adjoining rectangular blocks that present an edge perpendicular to the wind direction to be evaluated. Each of these blocks is used as a source area to compute the evaporation-condensation-diffusion process over the lake and surrounding areas of interest.

To evaluate the potential for steam fog and subsequent drift off the lake, an ambient air temperature, relative humidity, wind direction, wind speed, and atmospheric stability are input to the model for a given lake source area and water temperature. The model output is water vapor concentration at orthogonal grid points that cover the area of interest. A grid mesh of 500 meters is normally used, but was frequently varied to determine the location of critical values of water vapor concentration.

A2.3 References

1. R. R. Hippler, “Analytical Study of Cooling Pond Fogging at Low Temperatures for Commonwealth Edison Company,” The Research Corporation of New England, Wethersfield, Connecticut, 1972.

2. C. W. Thornthwaite and B. Holzman, “The Determination of Evaporation from Land and Water Surfaces,” Monthly Weather Review, January 1939.

3. T. F. Malone (Ed.), “Compendium of Meteorology,” American Meteorological Society, Boston, Massachusetts, P. 505, 1951.

4. D. B. Turner, “Workbook of Atmospheric Dispersion Estimates,” U.S. Dept. of Health, Education and Welfare, Public Health Service Publication No. 999-AP-26, p. 40, 1969.

5. T. F. Malone (Ed.), “Compendium of Meteorology,” American Meteorology Society, Boston, Massachusetts, p. 1180, 1951.

6. M, Kumai, “Arctic Fog Droplet Size Distribution and Its Effects on Light Attenuation,” U.S. Army Cold Regions Research and Engineering Laboratory, Hanover, New Hampshire, April 1972.

2.4 HYDROLOGIC ENGINEERING

2.4.1 Hydrologic Description

2.4.1.1 Site and Facilities

The Clinton Power Station (CPS) site is located 6 miles east of the city of Clinton, DeWitt County in central Illinois. The condenser cooling water is provided from the U-shaped cooling lake (Lake Clinton) that has been formed by construction of a dam just downstream from the confluence of North Fork of Salt Creek with Salt Creek. Figure 2.4-1 shows the site characteristics, structures, and facilities for the station and the cooling lake.

The cooling lake was formed by constructing an earth dam across Salt Creek 1200 feet downstream from its confluence with North Fork and 3300 feet upstream from Illinois State Route 10. The location of the dam is approximately 4 miles east of Clinton. The Salt Creek and North Fork fingers of the U-shaped lake extend 14 miles and 8 miles, respectively, upstream from the dam. The drainage area of the lake is 296 mi^2. The surface area of the lake is approximately 4900 acres (7.65 mi^2 - 2.6% of the drainage area) and the storage capacity is 74,200 acre-feet at a normal pool elevation of 690 feet. (All elevations referred to in this USAR are based on mean sea level (MSL) datum, U.S.G.S., 1929 adjustment.)

The station is located between the two fingers of the lake with a station grade elevation of 736 feet and plant floor elevation of 737 feet. The station is approximately 3-1/2 miles northeast of the dam and 1 mile south of U.S. Highway 54. The station circulating water screen house is located on the North Fork finger of the lake with the circulating water discharging back into the Salt Creek finger through a discharge flume as shown in Figure 2.4-1.

A concrete service spillway with an ogee type crest is provided on the west abutment of the dam to pass floods. An auxiliary spillway is provided on the east abutment to pass floods more severe than once-in-100-years recurrence including the probable maximum flood. A lake outlet structure is located near the west abutment to provide a minimum downstream release of 5 cubic feet per second (cfs). The ultimate heat sink (see Subsection 9.2.5) for the emergency core cooling system is provided within the cooling lake by constructing a submerged dam across the North Fork with an approach channel leading into the circulating water screen house.

The access to the station is from the Canadian National/Illinois Central Railroad and U.S. Highway 54. The ground topography along the station access route is favorably high and the grades have been located well above the probable maximum flood level in the lake. The station access road and railroad do not cross any stream and will not be affected by any flood conditions at the site. The access to the dam is provided from Illinois State Route 10.

Major highway and railroad bridges affected by the lake have been raised. The township road bridges affected by the lake have been removed or reconstructed and the roadway relocated.

The station area is provided with a drainage system which will drain into Salt Creek and the North Fork. The area traversed by the discharge flume is provided with drainage crossings into the Salt Creek finger of the lake.

Safety-related elevations are as follows:

a. normal groundwater table at station site - 675.0 to 729.0 ft

b. fuel building

   1. top of base slab - 712.0 ft
   2. adjacent grade - 736.0 ft
   3. building grade floor - 740.0 ft

c. containment building

   1. top of base slab - 712.0 ft
   2. lowest hatch - 737.0 ft

d. auxiliary building

   1. top of base slab - 707.5 ft
   2. adjacent grade - 736.0 ft
   3. building grade floor - 737.0 ft

e. diesel-generator and HVAC building

   1. top of base slab - 702.0 ft and 712.0 ft
   2. adjacent grade - 736.0 ft
   3. building grade floor - 737.0 ft

f. control building

   1. top of base slab - 702.0 ft
   2. building grade floor - 737.0 ft

g. radwaste building

   1. top of base slab - 702.0 ft
   2. adjacent grade - 736.0 ft
   3. building grade floor - 737.0 ft

h. circulating water screen house

   1. top of base - 657.5 ft
   2. adjacent grade - 698.0 ft

3. operating floor - 699.0 ft

4. top of watertight enclosure around Seismic Category I equipment - 730.0 ft

i. cooling lake

1. normal pool - 690.0 ft

2. 100-year flood (at station site) - 697.0 ft

3. 100-year flood (at main dam) - 697.0 ft

4. probable maximum flood (at station site) 708.9 ft

5. probable maximum flood (at main dam) - 708.8 ft

6. 100-year drought (low water) - 682.3 ft

7. crest of main dam - 711.8 ft

8. crest of service spillway - 690.0 ft

9. crest of auxiliary spillway - 700.0 ft

j. ultimate heat sink

1. bottom - 668.5 ft

2. crest of submerged dam - 675.0 ft

k. shutdown service water system outlet structure

1. invert of the pipes - 675.0 ft

2. top of base - 671.0 ft.

2.4.1.2 Hydrosphere

Salt Creek is located in the central region of Illinois and within the Sangamon River basin which drains into the Illinois River (References 1 and 2). Figure 2.4-2 shows the general hydrologic features and hydrographic network in the basin.

Figure 2.4-3, Sheets 1 through 4, are keyed to Figure 2.4-2 and show the topography of the area.

Salt Creek is the principal tributary of the Sangamon River. It rises 15 miles east of Bloomington in McLean County and flows in a southwesterly direction into DeWitt County. Thereafter, it pursues a westerly course to join the Sangamon River 8 miles east of Oakford in Menard County. The total length of Salt Creek is 92 miles and the drainage area is 1860 mi^2.

The cooling lake for the Clinton Power Station is located in the upper reaches of Salt Creek 28 miles from its source. The drainage basin above the dam site has a fan shape with an area of

296 mi^2. The highest elevation of the drainage basin is 910 feet and the lowest elevation at the dam site is 650 feet. The drainage basin consists of farm lands and pasture lands with trees abounding along the floodplains and adjacent areas.

A gauging station on Salt Creek is located near Rowell, 12 miles downstream from the dam site, with a drainage area of 335 mi^2. The station has records from October 1942 to 1976 (References 3, 4 and 5).

Table 2.4-1 gives the mean monthly runoff, rainfall, and natural lake evaporation data considered representative of the Salt Creek basin (References 3, 4, 5, 6, and 7).

At Rowell gauging station, the average discharge of Salt Creek for a period of 33 years is 241 cfs which is equivalent to 9.77 inches of rainfall per year. The maximum flow of record was 24,500 cfs (May 16, 1968) and minimum flow observed is 0.7 cfs (October 4, 1954) (References 4 and 5). Annual snowfall averages 22 inches (Reference 7).

There are no existing river control structures located upstream or downstream of the dam site which can affect the safety of the lake and station structures or the availability of water supply.

There are no known surface water users of Salt Creek or Sangamon River water that could be affected by accidental or normal releases of contaminants. Based on available information (References 8, 9, and 10) there is no municipal or private use of Salt Creek or the Sangamon River for drinking purposes. This is also true for the Illinois River into which the Sangamon River flows. The closest user of downstream water for drinking purposes is Alton, Illinois, on the Mississippi River, 242 river miles from the Clinton Power Station site.

There is no known usage of Salt Creek water for irrigation in DeWitt, Logan, Menard or Cass Counties (References 11 through 14). While irrigation farming does occur in this region, well water is the primary source of irrigation water. In DeWitt County, there are no irrigated farms.

Subsection 2.4.13.2 lists groundwater users in the vicinity of the station.

2.4.2 Floods

2.4.2.1 Flood History

Flood flows on Salt Creek recorded at Rowell gauging station (15 miles downstream from the station site) larger than 10,000 cfs are given in Table 2.4-2. The datum of gauge is 610 feet (References 1, 3, 4, and 5) above mean sea level.

A flood flow of 10,000 cfs has a return period of about 8 years based on Log-Pearson Type III frequency analysis made for 34 years of record (1943-1976) at Rowell gauging station. The discharge obtained for a flood of 100 year frequency is 29,900 cfs. The mean annual flood is 4300 cfs at a stage of about 20 feet.

Ice jam effects were recorded at the gauging station for floods observed during winter months, but the stages and discharges did not exceed the maximum observed values for the period of record.

2.4.2.2 Flood Design Considerations

The cooling lake is designed to withstand the effects of a probable maximum storm occurring over the entire drainage basin above the dam site.

Results of the hydrologic analyses discussed in Subsections 2.4.3 and 2.4.8 show that a probable maximum flood runoff into the lake routed through the spillways will raise the lake water level to elevation 708.8 feet at the dam site. The backwater effect along the North Fork finger will raise the probable maximum flood water level at the station site to elevation 708.9 feet. Superimposing the wind wave effect due to a sustained 40 mph wind acting on the probable maximum water level will result in wave runup elevations of 711.9 feet and 713.8 feet for significant waves and maximum (1%) waves, respectively, at the station site. The station's Seismic Category I structures with grade elevation of 736 feet will not be affected by the probable maximum flood design conditions. The circulating water screen house is designed to withstand the effects of probable maximum flood.

The maximum runup elevation at the dam for significant waves due to a sustained 40 mph wind acting on the probable maximum water level is elevation 711.0 feet. The top of the dam is at elevation 711.8 feet.

Precipitation data for selected major storms which have occurred in the Midwest and which are considered transposable to the region of Salt Creek basin, compared with the probable maximum precipitation used in the analysis, are shown in Table 2.4-3 (References 15 and 16).

There are two dams upstream of Clinton Lake on the North Fork of Salt Creek: Moraine View Dam on Dawson Lake and Vance Lake Dam on Clyde Vance Lake. The maximum combined storage capacity of these two reservoirs is 4446 acre-feet. This volume is small compared to the volume of Clinton Lake, 74,200 acre-feet at normal water level of 690 ft MSL. The effect of a flood wave resulting from a breach of these two dams coincident with a PMF event in the Clinton Lake watershed is not significant. (Reference 55)

Massive landslide from the valley walls into the cooling lake caused by a seismic disturbance is not possible because of lack of susceptible topographic and geological features. The geological conditions are described in Section 2.5. Thick glacial till available in the site precludes the possibility of massive landslides that can produce flood waves greater in magnitude than the probable maximum flood conditions and coincident wind wave effects.

Flooding due to tsunami is not possible at the CPS site.

Based on considerations and studies made, the probable maximum flood condition in the lake is considered the controlling event. All the safety-related structures are protected against this event.

2.4.2.3 Effects of Local Intense Precipitation

The effect of the local probable maximum precipitation (PMP) on the drainage areas adjacent to the plant including drainage from the roofs of structures was analyzed. It is conservatively assumed that the local surface drainage system would not function during the local PMP.

The analysis was made using the monthly 24-hour PMP estimates for Zone 7 from the U.S. Weather Bureau (USWB) Hydrometeorological Report No. 33 (Reference 16) as shown in Table

2.4-4. Since the August PMP is equal to the all-season high, it is used as the base for the other durations. The corresponding 48-hour PMP for the site area is 33.6 inches, which forms the design basis for the flood protection of the plant. The time distribution of the PMP into 6-hour periods was made using References 16 and 17, and is given in Table 2.4-5.

The maximum 6-hour rainfall is further divided into twelve 1/2 hour periods using the procedure described in pages 25-30 of Reference 18. The distribution is shown in Table 2.4-6. This is for a 6-hour interval only with the maximum rainfall of 43% for the seventh 1/2-hour interval equal to 24.48 inches in 6 hours. Distribution for 5-, 10-, and 15-minute durations was made using Table 3 of Reference 19. Using these values, summer PMP rainfall intensity-duration curve was obtained as shown in Figure 2.4-4.

In north-central Illinois (Reference 20) 70% of the annual snowfall occurs in December, January, and February. November, March, and April are normally the only other months when measurable snowfall occurs. Among these 6 months, the largest value for the monthly PMP is that for November equal to 14.9 inches for 200 mi^2 for a 24-hour period (Table 2.4-4). The corresponding 24-hour and 48-hour precipitation for the site area are 16.99 and 20.71 inches, respectively. Using the same procedure as for summer PMP, the winter PMP intensity-duration curve was obtained as shown in Figure 2.4-4, along with the summer PMP intensity-duration curve.

The immediate area around the plant building with the layout of roads, tracks, and drainage is shown in Figure 2.4-5. The areas surrounding the plant are graded to direct surface runoff away from the plant. The area was subdivided into zones for this analysis. Some ponding is expected in the areas enclosed by the roads and tracks near the plant. The times of concentration of these zones were estimated after taking into consideration the effect of ponding. The peak flow was computed using the rainfall intensity and rational formula. The runoff from the roofs of the plant building was also taken into consideration. After ponding up, water would flow over the tracks. It is conservatively assumed that the peak flow occurs after the water level reaches the top of the roads/tracks. The attenuation due to the ponding is neglected. It is assumed that the tracks would act as weirs discharging the flow away from the plant. The head over the weir was computed using the standard weir equation. The backwater effect was added to the head to obtain the water surface elevation near the plant building. The estimated maximum water surface elevation around the plant is lower than the plant floor elevation of 737.0 feet.

Only the southern half of the building is safety-related and also contains safety-related equipment; there is no safety-related equipment on the northern half of the building. The floor drain system would take care of the water that might enter the building. Therefore, the safety-related items would not be affected by the local PMP.

The roofs of safety-related structures are designed to withstand the snow and ice loads due to winter probable maximum precipitation over a 100-year recurrence interval antecedent snowpack. Conservatively assuming that the roof drains are clogged at the time of precipitation, the maximum accumulation of water on the roofs of safety-related structures is limited by the height of the parapet walls which is 16 inches plus the hydraulic head necessary for the water to flow over the parapet wall. The corresponding load on various roofs is then equal to the weight of water equivalent to 16 inches depth plus the weight due to a depth of water equivalent to the head over the walls which depends on the roof area and the contributing other roof areas.

The required head varies from 0.84 inches over turbine building roof to 3.74 inches over machine shop and radwaste building roof. The roofs of the safety-related buildings are designed to withstand the above loads.

It has been conservatively estimated that during the 48 hour PMP in the Unit 2 excavation, without the protecting effect of the surrounding earth berms (refer to Subsection 2.5.4.14.4) and without the benefit of any drainage out of the excavation, elevation of the impounded rainwater runoff will be 728 ft. This is lower than the 730 ft. elevation considered in the hydrostatic design of the Unit 1 exterior walls.

The exterior walls of Unit 1 have been designed to withstand the simultaneous effects of a hydrostatic head of 730 ft. and an SSE.

All openings in the Unit 1 exterior walls below the 730 ft. elevation are sealed and waterproofed.

The site grading and drainage are described in the drawings S03-1045 and S03-1100 through S03-1110, which have been submitted to the NRC under separate cover. Roof drains are connected to the underground storm sewer system with piping. The plant site drainage system is designed to pass the 10-year storm without any flooding of the adjacent area. During a 100-year storm there will be minor flooding of the roads in the plant site for a short duration. For the purposes of analyzing the effect of local probable maximum precipitation on the safety-related structures, it was assumed that the drainage system does not exist.

All openings in the Unit 1 building below grade level that lead into Unit 2 excavation are closed and waterproofed. Hence, runoff and drainage into excavation for Unit 2 does not have any effect on Unit 1 structures or its operation (Q&R 240.6).

2.4.3 Probable Maximum Flood (PMF) on Streams and Rivers

The probable maximum flood is an estimated flood that may be expected from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible in the region. Compliance to Appendix A of Regulatory Guide 1.59 is discussed in Section 1.8.

2.4.3.1 Probable Maximum Preciptation (PMP)

Seasonal variation of probable maximum precipitation over the 296 mi^2 drainage area of Salt Creek was obtained from the USWB Hydrometeorological Report No. 33, "Seasonal Variation of the Probable Maximum Precipitation East of 105th Meridian for Areas from 10 to 1000 Square Miles and Durations of 6, 12, 24 and 48 Hours," dated April 1956 (Reference 16). Monthly and all-season depth-duration data for the basin are given in Table 2.4-7 (Reference 16).

The dam site and the basin are located in Zone 7. The precipitation for the summer month of August is the most critical and is equal to the all-season value. The design probable maximum precipitation for each 6-hour duration is shown in Table 2.4-8.

The PMP value of 25.2 inches for a duration of 48 hours used in developing the PMF hydrograph was assumed to be distributed uniformly over the entire 296 mi^2 area in accordance with the proper areal correction factor given in the USWB Hydrometeorological Report No. 33 (Reference 16). No spatial distribution of this precipitation was considered on the basis that the drainage area is small and the approach suggested in the USWB Hydrometeorological Report No. 33 is appropriate.

The distribution of the 48-hour PMP into smaller intervals was based on the procedure given in the U.S. Army Corps of Engineers

Civil Engineering Bulletin No. 52-8, EM1110-2-1411, "Standard Project Flood Determinations" (Reference 17). There are two 24-hour rainfall periods, arranged as per the procedure of Reference 16. Each of the two 24-hour period rainfall is subdivided into four 6-hour periods. The sequence of the PMP is shown in Table 2.4-9.

A standard project storm (SPS) having precipitation equal to 50% of the PMP is considered to occur 3 days prior to the PMP as the antecedent storm in accordance with the recommendations given in Regulatory Guide 1.59. The procedures to develop the floods and flood routing are discussed in Subsections 2.4.3.4 and 2.4.3.5 respectively. The effect of maximum seasonal snow accumulation that coincides with the PMP in the winter months was studied. The weight of maximum snow on the ground plus the maximum probable snowstorm in the central Illinois region is estimated to be 40 psf (Chapter 10, Reference 18), or approximately 7 inches of water.

The maximum accumulation and the winter PMP of 14.4 inches (highest for the entire winter season) for the month of March would be 21.4 inches which is less than the August PMP.

2.4.3.2 Precipitation Losses

The topography of the Salt Creek basin is gentle to moderate. Using the soil maps published by the University of Illinois Agricultural Experiment Station, about 90% of the drainage area is found to be associated with Flanagan silt loam, Drummer clay loam, and Huntsville loam; the rest is sawmill clay load. The first three types of soils mentioned belong to hydrologic soil group B based on the U.S. Soil Conservation Service soil grouping (References 21 and 22). The initial loss depends upon antecedent moisture condition, in addition to other factors. From the data furnished by the U.S. Army Corps of Engineers (Reference 23), the initial loss is taken as 1.5 inches for the antecendent standard project storm and zero for the probable maximum precipitation.

Infiltration rates vary throughout the storm period from a high rate at the beginning, to a relatively low and uniform rate as the precipitation continues. Infiltration also depends on antecedent field moisture conditions, slope, soil type, vegetation, etc. Based upon the data furnished by the U.S. Army Corps of Engineers (Reference 23), the infiltration rate is assumed to be a constant equal to 0.1 in/hr.

The above values of initial loss and infiltration rate were used in calculating the rainfall excess for both the antecedent standard project storm and probable maximum precipitation. It is conservatively assumed that the initial loss is applicable only to the antecedent standard project storm and the infiltration is applied to both. Tables 2.4-10 and 2.4-11 show the rainfall excess values for the probable maximum precipitation and antecedent standard project storms respectively, for a 48-hour period.

2.4.3.3 Runoff and Stream Course Models

The unit hydrograph for Salt Creek at the gauging station near Rowell was derived from storm data recorded in 1943 and 1944. It is presented in the publication, "Unit Hydrographs in Illinois," by W. D. Mitchell (Reference 2). In the same publication, a synthetic method of deriving a unit hydrograph was developed. This method is applied in constructing the unit hydrographs. Figure 2.4-6 shows the synthetically developed unit hydrograph for Salt Creek at the dam site under natural river conditions compared with the Rowell Station unit hydrograph adjusted by direct ratio of drainage areas at the dam site and the gauging station. Figure 2.4-7 shows the unit hydrographs derived from the subbasin drainage areas above the dam site (Reference 2). The total drainage area at the dam site is relatively small. Flood hydrographs were developed for headwater areas and other subareas in the drainage basin that drain directly into the lake.

Flood routing through the proposed cooling lake was done using a "storage indication" routing procedure. The U.S. Army Corps of Engineers' Hydrologic Engineering Center computer program 22-j2-L210, "Spillway Rating and Flood Routing," was used in the computations.

A storage indication routing procedure is appropriate in this case because of the large depth of the lake (average 25-foot depth during floods).

This was further substantiated by the results of the backwater calculations. Under PMF conditions, the average slope of the lake surface was seen to be only about 0.06 ft/mi. The spillway rating curves are given in Figure 2.4-8.

Initial pool level was taken at elevation 690.0 feet for flood routing. This is the normal lake level and also the crest elevation of the service spillway. Elevation 690.0 feet was taken for the starting pool elevation because uncontrolled service spillway restores the pool to that elevation within a relatively short period following a major flood.

Runoff Model

Synthetic unit hydrographs were developed for subareas of Salt Creek and North Fork upstream of the dam site. The unit hydrographs were derived by using the information from Reference 2. Lag times have been computed according to the associated subareas.

Computation of Lag

An appropriate postulated relationship for Illinois streams is expressed by the formula: $t = 1.05\ A^{\circ.60}$, (Reference 2) in which A = drainage area in square miles and t = lag in hour.

The variation of maximum ordinate of synthetic unit hydrograph obtained by using the above formula for drainage areas upstream of the Rowell gauging station is only within 3% of the maximum ordinate of the unit hydrograph derived from observed flood hydrographs (Reference 2).

Duration of Unit Hydrographs

Ideal duration varies from one basin to another depending upon several characteristics of the basin; the most important is the size. For small subareas, durations of 1/2 hour and 1 hour were selected. Then, by using the S-curve hydrograph method, 2-hour unit hydrographs were obtained.

Ratio of Duration to Lag

This ratio determines which type of synthetic unit hydrograph should be used. Altogether nine forms for different basin sizes and ratio of duration to lag are given in Reference 2.

Unit Hydrograph Ordinates

The unit hydrograph ordinates were computed using the following procedure given in Reference 2.

$$q_s = \frac{A \text{ x nd}}{0.03719} \qquad (2.4\text{-}1)$$

Where q_s = total cfs-intervals

A = drainage area in square miles

and nd = average number of points in a 24-hour period.

Actual ordinates of unit hydrographs equal qs times value of distribution graph (given in nine associated forms).

All the pertinent information regarding the area, time lag, etc., of subareas of Salt Creek and North Fork along with the total area upstream of the dam site are given in Table 2.4-12. For lake area (8 mi^2), hydrograph ordinate corresponding to 1 inch of runoff has been computed for 2-hour duration.

Routing Coefficients

Channel routing procedure has not been used. Instead, flood hydrographs were developed for headwater areas and local areas that drain directly into the lake. Time lags between flood hydrographs of subareas have been computed based upon celerity of flood waves. The average depth of the lake during flood passage is about 25 feet. The distance from the dam site to the headwater area of Salt Creek is 14.2 miles (about 75,000 feet).

Celerity

The celerity of flood wave in feet per second is given by the formula

$$C = \sqrt{gy} \qquad (2.4\text{-}2)$$

Where

C = celerity in feet per second

g = acceleration due to gravity

and y = average depth of water in feet.

For an average water depth of 25 feet, the celerity is about 28 fps. The time lag for the flood wave to travel a distance of 75,000 feet is about 1 hour.

The flood hydrographs at the dam site are obtained by adding the hydrographs of the subareas taking into consideration the time lag.

The spillway flood routing was done using the U.S. Army Corps of Engineers computer program SPRAT (Spillway Rating and Flood Routing) for determining the flood elevations over the spillway.

2.4.3.4 Probable Maximum Flood Flow

The calculations for the probable maximum flood flow on Salt Creek at the dam site under natural river conditions are given in Table 2.4-13. The computations of the probable maximum flood inflow into the lake is given in Table 2.4-14. The peak probable maximum flood flow of Salt Creek under natural river conditions is 112,927 cfs. The peak probable maximum flood flow into the lake is 175,615 cfs. Figure 2.4-9 shows the hydrograph for the probable maximum flood inflow into the lake compared with the probable maximum flood hydrograph of Salt Creek at the dam site under natural river conditions.

There are two dams upstream of Clinton Lake on the North Fork of Salt Creek: Moraine View Dam on Dawson Lake and Vance Lake Dam on Clyde Vance Lake. The maximum combined storage capacity of these two reservoirs is 4446 acre-feet. This volume is small compared to the volume of Clinton Lake, 74,200 acre-feet at normal water level of 690 ft MSL. The effect of a flood wave resulting from a breach of these two dams coincident with a PMF event in the Clinton Lake watershed is not significant. (Reference 55) The cooling lake dam is designed to withstand the effects of a probable maximum flood and a coincident reservoir wind wave action. Spillways with uncontrolled crests are provided to pass floods. The dam and the spillways are protected against erosion due to wind wave action and flood flows.

2.4.3.5 Water Level Determinations

The maximum water level in the lake at the dam site was determined by routing the antecedent standard project flood (occurring 3 days prior to the PMF) and the PMF through the lake using the U.S. Army Hydrologic Engineering Center (HEC) computer program "Spillway Rating and Flood Routing" (Reference 24). The maximum water level obtained was elevation 708.8 feet with a peak outflow of 135,360 cfs passing through the spillways. The maximum water level at the station site was determined by making backwater calculations from the dam site to the station site along the North Fork finger of the lake, a distance of about 3-1/2 miles. The backwater computations were made using the U.S. Army (HEC) computer program, "Water Surface Profiles" (Reference 25), with a starting elevation of 708.8 feet which is the maximum water level at the dam during PMF. Eight cross sections plotted from the topographic map of the lake area with a scale of 1 inch = 400 feet, were utilized. Figure 2.4-10 (Sheet 1 of 5) shows the locations of the cross sections. The eight cross sections used in computing backwater profiles from the dam site to the station site along the North Fork are shown in Figure 2.4-10 (Sheets 2, 3, 4 and 5 of 5).

There is no gauging station upstream of the dam site. The nearest stream gauging station on Salt Creek is near Rowell, 12 miles downstream from the dam site. Historical flood levels at the Rowell gauging station were reconstituted using the slope area method. The roughness coefficients (n-values) used in backwater computations were verified by trying coefficients ranging from 0.020 to 0.040 for the main channel and from 0.035 to 0.068 for the floodplain. The values obtained for the roughness coefficients that best fit the historical flood stage are 0.030 and 0.050 for the main channel and floodplain, respectively. These coefficients were assumed to be applicable to the cooling lake PMF backwater computations. The creation of the cooling lake will raise the stage considerably; therefore, the roughness coefficients used are more than appropriate. Furthermore, the lake clearing substantially reduced the side channel roughness, which makes the choice of these roughness coefficients to be sufficiently conservative.

The values used for Manning's coefficient of roughness are 0.03 and 0.05 for the main channel and the floodplain, respectively. The backwater computations resulted in a maximum water level elevation of 708.9 feet at the station site. The computed PMF water surface profile is shown in Figure 2.4-11.

The excavated discharge flume is designed to carry a flow of 3057 cfs for two units (Subsection 2.4.8.2) with a maximum water surface elevation of 726.2 feet (power station grade is elevation 736 feet) and a minimum net freeboard of 3.8 feet.

Cross-drainage facilities are provided to drain the local areas. These facilities are designed for a once-in-a-100-year flood. Because the power station grade is much higher than the water surface elevations on the discharge flume, there is virtually no possibility that the water level will affect the safety of the power station.

2.4.3.6 Coincident Wind Wave Activity

The significant (33-1/3%) and maximum (1%) wave effects of a coincident 40-mph wind were superimposed on the probable maximum flood water level at the station site. The wave runups were calculated based on deep water and nonbreaking wave conditions with an effective fetch of 0.8 mile, a water depth of 40.5 feet, and the waves acting on a smooth 3:1 (horizontal to vertical) ground slope. The estimated wave runups are 2.95 feet and 4.85 feet for the significant waves and maximum (1%) waves, respectively. Superimposing the wave runup values on the probable maximum flood level at the station site resulted in a wave runup elevation of 711.9 feet for significant waves and elevation of 713.8 feet for maximum (1%) waves (References 27 and 28). The pressure distribution due to the waves is a combination of hydrostatic and hydrodynamic components and the exposed safety-related structures are designed to withstand these effects.

The maximum runup elevation at the dam for significant waves was calculated by superimposing the significant wave effects of a coincident 40-mph wind on the probable maximum flood water level at the dam site. The calculations are based on an effective fetch of 0.9 mile, a windtide fetch of 4 miles, a water depth of 58 feet, and an upstream slope of the dam of 3:1 (horizontal to vertical) with riprap. The wave runup obtained is 2.2 feet, resulting in a significant wave runup elevation of 711.0 feet at the dam site. Top of dam is at elevation 711.8 feet.

2.4.4 Potential Dam Failures, Seismically Induced

There are no existing dams upstream or downstream of the cooling lake which can affect the station safety-related facilities or the availability of the station cooling water supply. The U.S. Army Corps of Engineers and the Illinois Division of Water Resources were consulted and have indicated that there are no permit applications or proposals to construct dams on Salt Creek.

Furthermore, a postulated failure of the cooling lake dam will not result in the loss of water from the ultimate heat sink as discussed in Subsection 2.4.8.1.5.

2.4.5 Probable Maximum Surge and Seiche Flooding

There is no large body of water near the site where significant storm surges and seiche formations can occur. The size of the cooling lake is not large enough to develop surge and seiche flooding condition which is more critical than the probable maximum flood (PMF) condition.

2.4.6 Probable Maximum Tsunami Flooding

The station site will not be subjected to the effects of tsunami flooding because the site is not adjacent to a coastal area.

2.4.7 Ice Effects

The recording station on Salt Creek near Rowell is the only gauging station within the Salt Creek drainage basin. It has maintained a continuous streamflow record since October 1942. The records show intermittent ice effects during the winter months. Most of the recorded ice effects are minor in terms of the stage-discharge relationship, except for the ice jam that occurred on February 11, 1959. The maximum gauge height caused by this ice jam was 24.84 feet with a peak discharge of 7500 cfs. The datum of gauge is elevation 610 feet. Based on the ice-free stage-discharge relationship, the gauge height corresponding to a discharge of 7500 cfs is 22.14 feet (Reference 29). The ice jam effect raised the flood level by 2.7 feet. The streamflow records also show that the maximum recorded ice jam effects were less than the maximum flood stage and discharge values observed during the period of record.

The effects of ice formation and the probable maximum winter flood on the lake water level would be less than that of the probable maximum summer flood. Table 2.4-7 shows that the probable maximum precipitation for a duration of 48 hours for the month of August (25.2 inches) is greater than that for the month of February (13.8 inches) by 11.4 inches.

The average thickness of sheet-ice that could form in the lake area is estimated to be 10 inches, without considering the increase in lake water temperature due to station operation. The calculation is based on the empirical method developed by Assur (Reference 43), using a coefficient of snow cover of 0.85 and a value of 115 for the accumulated degree-days since freeze-up obtained from the average temperature data of Decatur, Urbana, and Springfield (Reference 44). The shutdown service water pumps are located in the screen house. The inlet to the screen house is at elevation 670 feet, 5 feet below the design water level of the ultimate heat sink, giving a water depth of 12.3 feet for station operation during lake low water level conditions. The occurrence of an estimated ice thickness of 10 inches in the intake area when the water level is at elevation 675 feet would not block the flow into the screen house. The availability of station cooling water will not be affected by ice formation in the screen house area. Additional protection against any probable ice blockage in the intake area is provided with the installation of a warming line at the inlet to the screen house, designed to maintain a minimum water temperature of 40°F during winter operaiton.

Ice formation or ice jams causing low flow conditions in the streams would not affect the performance of the ultimate heat sink due to its submerged condition. The ultimate heat sink is full and will be maintained.

2.4.8 Cooling Water Canals and Reservoirs

2.4.8.1 Cooling Lake

The cooling lake is designed to provide cooling water to the station and to remove the design heat load from the circulating water before the water circulates back into the station. The lake has a normal pool elevation of 690 feet with a surface area of approximately 4900 acres (7.65 mi^2) and a volume of 74,200 acre-feet. Figure 2.4-12 shows the lake area-capacity curves derived from the topographic map of the area with a scale of 1 inch equals 400 feet and a contour interval of 4 feet.

The lake started filling when the main dam was closed on October 12, 1977. Six months after dam closure, the lake level was observed at elevation 687.5 feet. The downstream releases through the lake outlet works during this period varied from 40 to 130 cfs.

The capacity of the lake was analyzed with a design drought condition of 100-year recurrence interval. The analyses of drought effects on the cooling lake are discussed in Subsection 2.4.11. The minimum lake water level obtained for the 100-year drought is elevation 682.3 feet. The lake will be able to withstand the effect of the 100-year drought without affecting normal station operations.

The loss of lake capacity due to sedimentation in the lake was analyzed. Based on the results of sedimentation surveys and studies conducted by the Illinois State Water Survey (Reference 30) on 85 reservoirs in Illinois, the annual capacity loss for a drainage area of 296 mi^2 and a lake capacity of 4.7 inches is 0.09% of lake capacity. Applying a maximum error factor of 60%, as suggested in Reference 30, results in a possible maximum capacity loss of 0.14% of lake capacity per year, or a sedimentation rate of 0.40 acre-feet/mi^2/yr.

Lake Bloomington, located 35 miles north of Lake Clinton, is one of the reservoirs included in the sedimentation surveys. Lake Bloomington is on Money Creek and has a watershed area of 61 mi^2. After an impoundment of 26 years (1929-1955), the average rate of sedimentation was observed to be 0.5 acre-feet/mi^2/yr (Reference 31).

Five water sampling stations were established in 1972 at the site of Lake Clinton. The turbidity measurements of the water obtained at the site indicate that the rate of sedimentation is less than 0.5 acre-feet/mi^2/yr.

On the basis of the Illinois reservoir sedimentation studies and the turbidity measurements at the site, a sedimentation rate of 0.5 acre-feet/mi^2/yr was used in the lake sedimentation analysis. The total sediment that will deposit in the lake for a period of 50 years will be 7400 acre-feet; a loss of 10% in lake capacity.

The cooling lake is designed to withstand the effects of a PMF. Subsection 2.4.3 discusses the effects of the PMF on the lake.

The scour and undermining of the upper portion of the soil-cement abutments, caused by high water, was repaired by pumping a sand cement grout into the undermined area. An expanded metal-forming material was used to contain the grout. A hand-excavated, cutoff trench will prevent the recurrence of the undermining. (See Figure 2.4-51 for further details. Repair of the south abutment was completed on October 22, 1981, and repair of the north abutment was completed on October 26, 1981. (Q&R 240.1)

2.4.8.1.1 Cooling Lake Dam

The plan of the cooling lake dam, spillways and outlet works is shown in Figure 2.4-13. The cooling lake dam is a homogeneous earthfill dam with a maximum height of 65 feet above the creek bed and a length of 3040 feet. The top of the dam is at elevation 711.8 feet. A freeboard of 3 feet is provided to prevent overtopping of the dam by the PMF and significant wave runup.

Both the upstream and downstream face of the dam have a side slope of 3:1 (horizontal to vertical). The upstream face is provided with an 18-inch thick riprap laid on two 9-inch layers of graded filter materials for protection against wind wave erosion and lake drawdown effects. The riprap is designed for 50 mph wind on lake normal pool. The downstream face is provided with seeded topsoil for protection against the erosive effect of rain falling over the dam. An 18-inch thick riprap laid on two 9-inch layers of graded filter materials is provided at the toe of the dam for erosion protection against tailwater effects. The riprap is designed for 50 mph wind on l00-year tailwater flood level (References 32 and 33).

The geotechnical design criteria for the main dam is discussed in Subsection 2.5.6.4. Figure 2.4-14 shows the typical section of the main dam. A cutoff into the Illinoian till and provision of a sand drainage blanket are made for seepage control under the dam and in the abutments.

A permit to construct the dam and appurtenant structures was granted by the Illinois Department of Transportation on April 7, 1976. The dam was closed on October 12, 1977. During dam closure, the creek flow was diverted into the lake outlet works. The water was discharged back to Salt Creek through the service spillway stilling basin and the discharge channel.

The probable maximum flood elevation is 708.8 feet and the maximum wave runup elevation is 711.95 feet. The top of the dam is at elevation 711.8 feet. The time duration over which overtopping by wave action would occur is only 2.5 hours. Any overtopping that would occur would be in the form of spray because the wave runs up the upstream slope and the water would be lifted into the air, thus creating a fine spray. Since most of the runup would be contained by the dam, only about 0.15 feet of the runup would be overtopping in the form of spray. The downstream slope is well protected with grass against gully erosion due to rain and, hence, any overtopping that might occur for a period of only 2.5 hours would not cause any significant damage to the downstream slope of the dam.

Standard dam engineering practice, which recognized the overtopping of a dam by a few extreme waves, indeed, provides for protecting the dam against wave overtopping only up to the significant wave (rather than the maximum wave), as may be noted from the following:

1. U.S. Army Corps of Engineers' recommended practice regarding freeboard is quoted hereunder from their Engineer Circular No. 1110-2-27 dated February 19, 1968.

> "The basic objective does not require zero overwash of the embankment by occasional waves under extreme surcharge conditions, but only that such occurrences should not be of such magnitude and duration as to threaten the safety of the dam...Accordingly, it is current practice to estimate freeboard allowance for wave action above the maximum reservoir surcharge level as being equal to the computed height of runup of the significant wave (H_s) as computed from adopted design wind criteria. This criteria is based on the assumption that the overwash resulting from waves exceeding H_s (which include

> approximately 13 percent of the total number of waves in a spectrum) would not endanger the integrity of the dam or otherwise cause significant damage to warrant provision of higher freeboard allowances... Moreover, the short duration of maximum or near maximum surcharge levels in such cases would limit the duration of wave overwash that the embankment would be required to withstand.

2. U.S. Bureau of Reclamation (U.S.B.R.) recommends a freeboard of 3 feet for fetches less than 1 mile as is the case for Clinton Dam. (Design of Small Dams, page 274, U.S.B.R., 1974.) (Q&R 240.7)

2.4.8.1.2 Service Spillway

A service spillway is provided to pass a design flood of 100-year frequency with a flood water surface elevation of 697 feet in the lake. It is located on the west abutment of the dam mainly due to favorable soil conditions. The service spillway is an uncontrolled concrete ogee type, semicircular in plan, with a crest length of 175 feet and a crest elevation of 690 feet. The height of the concrete ogee is 10 feet. From the ogee section, the water will discharge through an 80-foot wide concrete chute and into a stilling basin. A discharge channel is excavated to convey the water from the stilling basin to the main channel of Salt Creek.

The location of the service spillway is shown in Figure 2.4-13. The plan and details are shown in Figures 2.4-15 and 2.4-16. The total length of the spillway from the face of the ogee section to the end of the stilling basin is 603 feet.

The crest shape of the ogee is based on the equation

$$X^{1.85} = 2\left(H_d^{0.85}\right)Y \qquad (2.4\text{-}3)$$

where H_d is the design head (Reference 34). A design head of 12 feet is used which corresponds to a head over the crest resulting from a standard project flood. The peak discharge through the spillway for the 100-year flood is 11,450 cfs. The velocity on the spillway crest is 12.9 fps and the water surface elevation downstream of the ogee is 687.6 feet. The peak discharge through the spillway for the PMF is 33,200 cfs with a flood water surface elevation of 708.8 feet at the crest. The velocity on the crest is 18.2 fps and the water surface elevation downstream of the ogee is 696.5 feet. The spillway rating curve is shown in Figure 2.4-8.

The chute section is designed considering the natural ground profile and the economics of the structure. It consists of a sloping channel of two different slopes (0.824% and 2.98%) and an inclined drop with a slope of 2.5:1 (horizontal to vertical), terminating into a horizontal stilling basin. An underdrainage system is provided to reduce the uplift and avoid piping conditions. It consists of graded gravel and sand materials with perforated pipes and weep holes located at selected points along the chute.

The horizontal stilling basin is designed on the basis of the U.S. Bureau of Reclamation practices. A type II basin is adopted with the apron at elevation 637 feet. The 100-year flood discharge has a Froude number of 6.1 and hydraulic jump sequent depths of 2.6 feet and 20.9 feet. The PMF discharge has a Froude number of 4.2 and hydraulic jump sequent depths of 6.7 feet and 36.6 feet. The tailwater elevations for the 100-year flood and the PMF are 660 feet and 678 feet, respectively.

Riprap is provided downstream of the stilling basin for a distance of 80 feet as protection against erosion. The riprap is 2 feet thick laid on 1-foot thick gravel filter materials. The riprap could withstand a maximum velocity of 10 fps.

The top of the retaining walls in the chute section is provided with a minimum freeboard of 1 foot above the PMF water surface profile. The top of the retaining walls for the stilling basin is provided with a freeboard of 4.5 feet above the standard project flood level. The backfill and graded area adjacent to the stilling basin are provided with riprap and seeded topsoil for erosion protection.

The 100-year flood level in the lake is the basis for determining the auxiliary spillway crest elevation. The auxiliary spillway is designed to function only during floods greater than the 100-year flood. The crest of the auxiliary spillway is set at elevation 700 feet for the 100-year flood flow to discharge entirely through the service spillway.

The 100-year storm precipitation data were obtained from the U.S. Weather Bureau publication, "Rainfall Frequency Atlas of the United States." The 100-year rainfall for various durations and storm distribution are given in Tables 2.4-15, 2.4-16, and 2.4-17 (References 19 and 35).

The rainfall excess was estimated by analyzing the soil type and land usage in the basin and following the procedure outlined in the U.S. Bureau of Reclamation's publication, "Design of Small Dams" (Reference 35). The calculations for the hydrologic soilcover complex number for soil group B are shown in Table 2.4-18. Using a soil antecedent moisture condition III which considers the soil as nearly saturated, the rainfall excess and the infiltration losses for the 100-year storm were determined using the runoff curves in Appendix A of Reference 35. Table 2.4-19 shows the 100-year rainfall excess and infiltration losses.

The 100-year flood hydrograph was computed using the procedure in the U.S. Army Corps of Engineers Manual, "Flood Hydrograph Analyses and Computations" (Reference 36). The computations are based on the unit hydrographs shown in Figures 2.4-6 and 2.4-7 and the rainfall excess values given in Table 2.4-19. Figures 2.4-17 and 2.4-18 show the 100-year flood hydrographs for the inflow and outflow from the lake and for the natural river conditions. The 100-year flood peak discharge at the dam site under natural river conditions is 21,670 cfs, based on the unit hydrograph developed for the basin. The peak of the 100-year flood discharge into the lake is 32,600 cfs.

The 100-year flood level in the lake was determined by flood routing calculations using the SPRAT program (Reference 24). The 100-year flood water level is elevation 697 feet with a peak outflow at the service spillway of 11,450 cfs.

The tailwater rating curve for Salt Creek downstream of the dam is shown in Figure 2.4-19. The curve was developed by making backwater calculations starting at the creek section 4000 feet downstream from the Illinois State Route 10 bridge. The starting water surface elevations were calculated using the slope-area method. The cross sections of Salt Creek including the Illinois State Route 10 bridge section are shown in Figure 2.4-20. The locations of these sections are shown in Figure 2.4-10 (Sheet 1 of 5).

2.4.8.1.3 Auxiliary Spillway

The auxiliary (emergency) spillway is located east of the dam. The location is chosen on the basis of obtaining a better approach condition. The auxiliary spillway is designed to pass floods more severe than the 100-year flood and up to and including the PMF with SPF as antecedent flood. The spillway provides protection to the dam against overtopping. The spillway is an open-cut type with a crest length of 1200 feet and a crest elevation of 700 feet. The flood water will be discharged back into the main channel of Salt Creek between the dam and the Illinois State Route 10 bridge. The location and plan of the auxiliary spillway is shown in Figure 2.4-13.

The peak discharge through the auxiliary spillway during the PMF with SPF as the antecedent flood is 102,800 cfs with a corresponding water level in the lake of elevation 708.8 feet. The maximum velocity at the crest is 14 fps. The crest control section consists of 9-inch thick asphalt concrete laid on 16 inches of compacted aggregate materials. The width of the asphalt concrete crest is 25 feet. Concrete cutoffs and riprap are provided upstream and downstream of the asphalt concrete crest to protect the crest against scouring. A 6-foot deep rock trench is provided at the end of the downstream riprap which varies in distance from the crest from 150 feet at the far end to 300 feet at the area near the dam. Figure 2.4-21 shows the auxiliary spillway sections and details.

The approach channel is excavated to elevations varying from 690 feet to 695 feet. The length of the approach channel is 1510 feet along the centerline of the spillway. The bottom of the discharge channel is elevation 695 feet. The length of the discharge channel is 2120 feet along the centerline of the spillway. The channels are provided with an erosion resistant soil with Bermuda grass cover which can withstand a velocity of 8 fps (Reference 26). Erosion control measures on the auxiliary spillway are provided for the safety of the dam and the spillway structure during extreme flood conditions.

2.4.8.1.4 Outlet Works

The lake outlet works is located on the west abutment of the dam, 160 feet east of the service spillway. The location of the outlet works is shown in Figure 2.4-13. The lake outlet works is provided primarily to release a minimum flow of 5 cfs to the creek downstream of the dam. The purpose of the minimum reservoir release of 5 cfs is to satisfy commitments made in the Final Environmental Statement. The discharge from the lake outlet works at normal pool elevation, with all the gates fully opened, is 170 cfs.

The plan, section, and details of the lake outlet works are shown in Figure 2.4-22. The lake outlet works consists of a submerged concrete intake structure of the drop inlet type, a 36-inch diameter precast, prestressed concrete entrance pipe, a wetwell type concrete control house with three cast iron sluice gates at different levels, and a 48-inch diameter precast, prestressed concrete outlet pipe terminating at the spillway stilling basin.

The crest of the intake structure is at elevation 668 feet with an inlet diameter of 84 inches, transitioning into a 36-inch diameter vertical section (throat). The crest elevation is determined based on the hydraulic requirements and recommendations given in the U.S. Bureau of Reclamation's "Design of Small Dams" (Reference 37). The inlet is provided with a trash rack and a vortex breaker. A provision for placing stop logs is made at the inlet of the entrance pipe for inspection or maintenance of the control gates.

The three cast iron sluice gates at the control house regulate the downstream releases of water from the lake. Two gates are 12 inch by 12 inch size with the centerline of one gate at elevation 686 feet and the other centerline at elevation 684 feet, and one gate 24 inch by 36 inch size located at the bottom of the control house at elevation 650.88 feet. The upper 12 inch by 12 inch gate will remain open during normal operating conditions. When the lake level falls to elevation 687 feet, the lower 12 inch by 12 inch gate will be opened. The bottom 24 inch by 36 inch gate will be opened when the lake level falls to elevation 685 feet. The gates are manually operated from the top of the control house. Locking devices are provided for the gates to prevent unauthorized personnel from operating the gates. A 15-foot wide concrete bridge is provided for access from the top of the dam to the control house.

The 48 inch outlet pipe is located below a good natural soil formation and is provided with concrete anti-seep collars to prevent seepage problems in the body of the dam. The outlet pipe will discharge the water into the stilling basin of the service spillway where the energy of flow will be dissipated. The discharge channel downstream of the stilling basin will convey the flow to the main channel of Salt Creek.

The lake outlet works was used to discharge the Salt Creek flow during final dam closure. A diversion channel was constructed from the main channel of Salt Creek to the lake outlet works. A construction inlet structure was connected to the bottom of the outlet works intake structure by a 36-inch concrete pipe. The pipe was plugged and grouted after the construction of the dam and after the lake level had reached the design low water elevation.

2.4.8.1.5 Flow Through the Ultimate Heat Sink

The ultimate heat sink is a submerged pond formed by the construction of a submerged dam across the North Fork channel. The submerged dam is located 1 mile west of the screen house. The location of the ultimate heat sink is shown in Figure 2.4-1. Subsection 9.2.5 describes the submerged ultimate heat sink. Figure 2.4-23 shows the plan of the ultimate heat sink. The cross sections through the ultimate heat sink, submerged dam, and baffle dike are shown in Figure 2.4-24.

The top of the submerged dam is at elevation 675 feet with a width of 30 feet and a length of 2350 feet. The dam consists of homogeneous compacted backfill materials with a side slope of 5:1 (horizontal to vertical) on both faces of the dam. The excavation for foundation of the dam is extended to the Illinoian till. A 2-foot thick compacted soil-cement is provided at the top and side slopes of the dam and extends into a horizontal apron downstream of the toe of the dam. The toe is at elevation 670.3 feet. A random fill is provided to elevation 673.5 feet at the end of the soil-cement apron to a distance of 290 feet from the centerline of the dam. The random fill is placed in areas where the existing grade is lower than elevation 673.5 feet to create a stilling pool downstream of the dam. The baffle dike consists of homogeneous compacted backfill materials with a 30 foot width at the top and a side slope of 5:1 (horizontal to vertical) on both faces of the dike. The top of the baffle dike is at elevation 676 feet and is provided with a 3-foot thick compacted soil-cement. The bottom of the dike is founded on the Illinoian till. The length of the baffle dike is 3300 feet.

The area of the ultimate heat sink at the design water surface elevation of 675 feet is 158 acres with a total volume of 1067 acre-feet. The area-capacity of the ultimate heat sink is shown in Figure 2.4-25.

The velocities over the submerged dam were analyzed for flow conditions resulting from a sudden breach in the main dam. The breach was conservatively assumed to occur at the time of the PMF with the lake water level at elevation 708.8 feet. An instantaneous breach 100 feet wide and extending down to the Salt Creek bed (elevation 655 feet) was postulated. The outflow discharge and stage rating curves of the breach and spillway structures were developed using the method adopted by the U.S. Army Corps of Engineers (Reference 38).

The ultimate heat sink and the lake area downstream of the submerged dam were analyzed to evaluate the flow conditions due to the breach. The conventional level pool reservoir routing method was used. This method is based on the premise that the inflow volume minus the outflow volume equals the change in storage. The inflow volume is the flow through a control section, which is the constriction due to a railroad bridge upstream of the ultimate heat sink. The water surface elevations upstream and downstream of the submerged dam were determined from the flood routing calculations. Based on the water surface elevations and the corresponding flow over the submerged dam, the maximum velocities at the crest and at the toe of the dam were calculated. The maximum velocities obtained are 3.8 and 11.8 fps at the crest and at the toe of the dam, respectively, occurring 43 hours after the main dam breach. Correspondingly, a maximum velocity of 1.2 fps on the face of the baffle dike is obtained. The maximum velocity on the baffle dike is lower than the maximum velocity at the crest of the submerged dam because the induced flow is primarily parallel to the baffle dike. Shown in Figure 2.4-26 is the velocity profile at the crest of the submerged dam and on the baffle dike at various times after the main dam breach.

The velocities over the submerged dam and baffle dike were also analyzed with the occurrence of a PMF on the North Fork when the lake level is at the 100-year drought water surface elevation of 682.3 feet. The variation of lake level with time was determined with the PMF hydrograph of North Fork (Figure 2.4-27) as the inflow into the lake. The results are presented in Figure 2.4-28. Flow velocities over the submerged dam and baffle dike were determined with time and the results are presented in Figure 2.4-29. The maximum velocities obtained over the submerged dam and baffle dike are 2.1 and 2.6 fps, respectively.

The compacted soil-cement provided over the surface of submerged dam and baffle dike will protect these structures against the erosive effect of the velocities and flow conditions due to the postulated dam breach and the occurrence of a probable maximum flood on North Fork coincident with the 100-year drought lake water level of elevation 682.3 feet. Tests were made to determine the ability of soil-cement to prevent scouring along open flumes by the Civil Engineering Department of Oklahoma A&M College in 1942 (Reference 39). The tests consisted in permitting clean water, at a velocity of 28 fps, to flow continously for 6 days in an open flume with 4 1/2-inch thick compacted soil-cement lining. The results of the test show no appreciable wear or erosion on the soil-cement lining. Studies were made by W. G. Holtz and F. C. Walker of the U.S. Bureau of Reclamation in 1962 on the use of soil-cement as slope protection for earth dams (Reference 40). The results of the studies indicated that compacted soil-cement has a favorable performance as erosion protection against wind wave forces.

During the excavation for the ultimate heat sink and the construction of the submerged dam and baffle dike, the creek flow was diverted into an excavated channel along the north slopes of the ultimate heat sink. Figure 2.4-30 shows the plan and details of the diversion scheme for the construction of the ultimate heat sink. The diversion channel has a bottom elevation of 664 feet with a width of 50 feet and side slopes of 7:1 (horizontal to vertical) on the north side and 2:1 on the south side of the channel. A slurry trench extending to the Illinoian till was provided between the diversion channel and the ultimate heat sink to control seepage flow into the construction area of the heat sink. An earthen cofferdam was also constructed at a distance of 200 feet from the submerged dam to block the creek flow. No flooding problem was encountered during the construction of the ultimate heat sink and the submerged structures. The submerged dam closure was done on October 17, 1977, 5 days after final closure of the main dam. After a period of 1 month, the ultimate heat sink was filled to elevation 675 feet. During the filling period, flood flow did not occur on North Fork.

2.4.8.2 Station Discharge Flume

The discharge flume is provided to convey the plant discharge from the circulating water pipe discharge structure into the Salt Creek finger of the lake. The flume is located east of the plant area and runs due east toward the lake. The location of the flume provides an effective cooling surface area of 3650 acres in the lake. Figure 2.4-1 shows the location of the flume. The discharge flume is designed for a maximum flow of 3057 cfs with a nonscouring velocity of 1.5 fps. The flume has a bottom width of 120 feet and a side slope of 3:1 (horizontal to vertical). The total length of the flume is 3.4 miles (18,040 feet). A minimum freeboard of 3.8 feet is provided in the flume. A 6-inch thick crushed stone layer is provided on the side slopes of the flume for protection against erosion due to wind wave action in the flume. Riprap is provided on the lake side of the embankment fills for protection against erosion due to wind wave action in the lake.

Drop structures of the baffled apron type are provided at two locations along the flume to adapt the flume design to ground topography and to prevent scouring in the flume during station operations at design drought conditions in the lake. The two drop structures have the same width of 70 feet; the first one is designed for a drop of 18 feet, and the second is designed for a drop of 26 feet. Provisions against erosion are provided at the end of both structures.

Drainage crossings under the flume are provided at two locations to drain the areas north of the flume. The drainage structures consist of corrugated metal pipes designed for 100-year flood conditions. Anti-seep collars and erosion protection are provided on the structures.

A breach in the station discharge flume embankment is postulated to occur at the first crossing with a small creek 4800 feet from the circulating water pipe discharge structure. At the location of the postulated flume breach, the water in the flume will flow into the natural course of the creek in the southerly direction leading into the Salt Creek finger of the lake. This point of discharge into the lake is 3-1/2 miles downstream of the discharge flume outfall location. This condition will shorten the path of circulating water by 6 miles and reduce the effective cooling surface area by 1100 acres.

The occurrence of a breach in the flume can be detected through normal preventive maintenance inspections of the discharge flume and by periodic monitoring of the lake temperature recording instruments at the intake, discharge, and selected locations in the lake. The plant may be shutdown to ensure station safety will not be affected, if warranted by the degraded flume conditions.

2.4.9 Channel Diversions

There is no historical evidence of channel diversion of Salt Creek and North Fork of Salt Creek upstream of the dam site. The dam site is located on the upper reaches of Salt Creek, 28 miles from its source. The topographic characteristics and geological features of the drainage basin indicate that there is no possibility for the occurrence of a landslide that will cut off the streamflow into the lake. The history of ice jam formation discussed in Subsection 2.4.7 did not show evidence of flow diversion during winter months.

2.4.10 Flooding Protection Requirements

The flooding effects of a probable maximum flood (PMF) on Salt Creek and a local probable maximum precipitation (PMP) on the plant area are the design bases for flood protection of all station safety-related facilities. The considerations for selecting the PMF on Salt Creek as the design flood are discussed in Subsection 2.4.2.2. The effects of the PMF and a coincident wind wave activity on the lake at the station site are discussed in Subsection 2.4.3.

The maximum (1%) wave runup elevation at the station site is 713.8 feet, produced by a sustained 40 mph overland wind acting on the PMF still water elevation of 708.9 feet. The station grade elevation of 736 feet is 22.2 feet above the maximum wave runup level and 27.1 feet above the PMF water level. The safety-related facilities in the station area would not be affected by the PMF conditions in the lake. The only station facility that would be affected by the PMF is the circulating water screen house, for which the flood protection requirements are discussed in this section.

Wind wave forces caused by a sustained 48 mph overland wind coincident with the PMF pool and 67 mph overload wind coincident with normal pool were considered in the design of the circulating water screen house. Both breaking and nonbreaking wave forces were considered.

Flood protection for the safety-related systems and components in the circulating water screen house are provided to elevation 714 feet. The following protection measures are adopted to waterproof the compartments housing the safety-related systems and components (also see D3.6.4):

a. Water stops are provided in all construction joints up to elevation 714 feet.

b. Water seal rings are provided for all penetrations in exterior walls below elevation 714 feet.

c. Watertight doors are provided for all doorways located on both the entrance walls and the internal walls of the SSW pump rooms which are below elevation 714 feet. The watertight doors leading into the shutdown service water pump compartments are normally kept closed. Watertight doors SDI-9 and SDI-10 are normally kept closed, with SDI-9 chained on the interior side of the compartments and SDI-10 chained on the exterior side of the compartment. Administrative procedures require that plant personnel tour the circulating water screen house periodically to ascertain that systems, equipment, and structures are functioning properly (Q&R 240.04).

d. Hatches are provided on the roof of the essential service water pump structure (elevation 730 feet) for access during PMF.

The maximum water surface elevation due to a local PMP is 736.8 feet in the immediate station area where safety-related facilities are located. The flooding effects of a local PMP are discussed in Subsection 2.4.2.3. With the station floor elevation at 737 feet, the safety-related facilities would not be affected by the local PMP.

The eroded area south of the screenhouse was first graded and shaped to approximately a 3-to-1 slope. The revetment mat is composed of a double layer of industrial grade nylon fabric which is anchored to the slope by the use of a 2-foot deep trench. Into this mat, a sand/cement mortar is pumped to complete the installation of the revetment on the slopes above the normal pool elevation water line. (See Figure 2.4-52) (Q&R 240.5)

2.4.11 Low Water Considerations

2.4.11.1 Low Flow in Salt Creek

A design drought of 100-year recurrence interval is used in the determination of the minimum water level in the cooling lake. The 100-year drought runoff data for Salt Creek at the Rowell gauging station were derived from the low flow recurrence curves in the Illinois State Water Survey publication, "Low Flows of Illinois Stream for Impounding Reservoir Design" (Reference 30). Table 2.4-20 lists the low flows for 100-year drought for various durations.

The lake drawdown analysis for the 100-year drought was done starting at elevation 690 feet, using the net lake evaporation data given in Table 2.4-21, the forced lake evaporation data given in Table 2.4-22, a constant downstream release of 5 cfs, and an assumed seepage loss of 0.5% of lake capacity per month.

The minimum lake water level obtained for the 100-year drought with the station operating at 70% load factor is elevation 682.3 feet. The circulating water system is designed to operate with a minimum lake water surface elevation of 677 feet. This level is 5.3 feet below the low lake water level during a 100-year drought and therefore, the station cooling water supply will not be affected by a drought as severe as a 100-year drought.

In the event of the occurrence of a drought more severe than a 100-year drought that will bring down the lake water level to elevation 677 feet, station shutdown operations will be followed with the ultimate heat sink supplying water for the shutdown service water system. The ultimate heat sink is a submerged pond within the cooling lake formed by the construction of a submerged dam. The top of the submerged dam is at elevation 675 feet, which is the design water level for the ultimate heat sink. The performance of the ultimate heat sink will not be affected by low flow conditions in the streams.

2.4.11.2 Low Water Resulting from Surges, Seiches, or Tsunami

Surges, seiches, or tsunami conditions are not possible to occur and affect low water conditions in the lake and the ultimate heat sink because there is no large body of water near the site.

2.4.11.3 Historical Low Water

The worst drought of record in central Illinois occurred in 1954. Runoff data of Salt Creek at Rowell gauging station for the historic drought of 1952-1957 are given in Table 2.4-23 (Reference 4). The severity of the historic drought is found comparable to that of a drought with a recurrence interval of 50 years. The derived low flows for 50-year drought are given in Table 2.4-24. The comparison of the runoff values for the historic drought and the 50-year drought are given in Table 2.4-25.

The lake drawdown analyses for the historic drought and the 50 year drought were done using the 50-year recurrence interval evaporation values given in Table 2.4-21 for both drought conditions. The forced evaporation data, seepage losses and reservoir releases are the same as those used in the lake drawdown analysis for the 100-year drought. The lake low water level obtained is elevation 685.5 feet for both the historic and 50-year drought conditions.

2.4.11.4 Future Controls

Based on inquiries made with state and federal regulatory agencies, there are no future plans to use Salt Creek water upstream of the cooling lake. Any future use of Salt Creek water upstream of the site would not affect the availability of shutdown cooling water supply due to the submerged condition of the ultimate heat sink.

2.4.11.5 Plant Requirements

The estimated station water requirements for various operating conditions are shown in Table 2.4-26. The required minimum safety-related cooling water flow is approximately 12,300 gpm for Division I and II and approximately 1,000 gpm for Division III for an accident in Unit 1. The plant service water system and the circulating water system are not safety-related.

The circulating water screen house is described in Subsection 3.8.4. The arrangement and configuration of the circulating water screen house is shown in Drawing M01-1116. The figure shows the location of the circulating water pumps, plant service water pumps, shutdown service water pumps and fire pumps within the screen house. The intake to the screen house is at elevation 670 feet. The circulating water and service water pump suction sumps are all flat, with an invert elevation of 657.5 feet. The minimum design operating level for the shutdown service water pumps is elevation 671.5 feet, 2.8 feet below the lowest level of the ultimate heat sink after 30 days of operation as shown in Table 9.2-15. The design bases for the shutdown service water system and the ultimate heat sink are discussed in Subsections 9.2.1 and 9.2.5.

The minimum lake water level for the 100-year drought is elevation 682.3 feet. The minimum design operating level for the circulating water pumps and the plant service water pumps is elevation 677 feet and elevation 672 feet, respectively. The cooling water supply for normal station operation will not be affected by the 100-year drought condition. The design bases for the plant service water system are given in Subsection 9.2.1. The circulating water system is discussed in Subsection 10.4.5. In the event of a severe drought that will reduce the lake water level to elevation 677 feet, station shutdown operation will be followed. The design water level for the shutdown service water pumps and the ultimate heat sink are not affected by drought conditions in the lake.

The permit to construct the main dam and impound the water of Salt Creek for a cooling lake was granted on the basis of providing a minimum downstream release of 5 cfs. The station cooling water supply and lake drawdown analysis is based on satisfying the permit requirements for downstream releases.

2.4.11.6 Heat Sink Dependability Requirements

The cooling lake is the source of cooling water supply during normal operation. The design considerations and description of the cooling lake and the main dam are discussed in Subsection 2.4.8. In the unlikely event of a failure of the main dam and complete loss of the cooling lake, the submerged ultimate heat sink will supply the cooling water for emergency station operation.

The ultimate heat sink is a submerged pond within the cooling lake formed by the construction of a submerged dam across the North Fork channel. The ultimate heat sink is adjacent to the circulating water screen house where the shutdown service water pumps are located. The cooling water will be pumped from the ultimate heat sink to the station by the shutdown service water pumps. The shutdown service water system discharge pipes will convey the water back to the ultimate heat sink through a discharge structure located 600 feet south of the circulating water screen house. A submerged baffle dike is provided between the discharge structure and the screen house to provide a longer cooling time for the circulating water.

The ultimate heat sink is designed to provide sufficient water volume and cooling capability for station shutdown operation for a period of at least 30 days and beyond, if necessary, without requiring makeup water. The design bases, description, and analysis of the ultimate heat sink are discussed in detail in Subsection 9.2.5. The plan of the ultimate heat sink is shown in Figure 2.4-23. Figure 2.4-24 shows the cross sections through the ultimate heat sink, submerged dam, and submerged baffle dike.

In addition to the water requirements for station shutdown operation, a minimum of 900,000 gallons (2.8 acre-feet) of water in the ultimate heat sink is made available for fire protection requirements. The fire protection system is described in Subsection 9.5.1. The basic fire protection requirements for nuclear power plants are given in a document published by NEPIAMAERP entitled, "Basic Fire Protection for Nuclear Power Plants," dated March 1970. This document states that fire pumps are "...to take suction from acceptable natural bodies of water or a minimum of two 300,000-gallon capacity suction tanks, automatically filled from a supply capable of filling one tank in eight hours." The submerged ultimate heat sink satisfies this requirement since it remains full of water. The gross area-capacity curve of the ultimate heat sink is shown in Figure 2.4-25. The volume available for sedimentation is based on the gross area-capacity minus the minimum cooling water requirement, and allowances for Fire Protection and liquefaction as reflected in Table 9.2-12.

In the vicinity of the ultimate heat sink, the groundwater table will rise to the level of the water surface in the cooling lake during normal operating conditions. In the event of losing the cooling lake, groundwater flow will be toward the sink from the higher groundwater table created by the lake. The calculated loss of water from the sink due to seepage through the submerged dam and its foundation is 0.003 cfs (i.e. insignificant).

The shutdown service water pumps and the fire pumps are located in the circulating water screen house. The circulating water screen house is designed to withstand the effects of a PMF and coincident wind wave forces caused by a sustained 48 mph overland wind and the effects of normal pool with waves caused by 67 mph overland wind speed. The design conditions for the circulating water screen house are discussed in Section 3.4 and Subsection 3.8.4.1. The shutdown service water system is discussed in Subsection 9.2.1. The effects of a sudden breach in the main dam on the ultimate heat sink, submerged dam, and baffle dike were analyzed. The analysis of the flow conditions through the ultimate heat sink due to a postulated 100-foot wide main dam breach are presented in Subsection 2.4.8.1.5. The results of the analysis show that the submerged dam and baffle dike could withstand the effects of the postulated dam breach and the design capacity of the ultimate heat sink would not be affected. The geotechnical design considerations for the submerged dam and baffle dike are discussed in Subsections 2.5.5.2 and 2.5.6.4.

The cooling lake water level in the vicinity of the ultimate heat sink will be monitored with a visually read stage level indicator. This gauge is installed at the circulating water screen house and it is read by station operating personnel. Real time water level at the dam is available on the US Geological Survey (USGS) website.

The design water level for the ultimate heat sink is elevation 675 feet. The minimum design operating level for the circulating water pumps is elevation 677 feet, 2 feet higher than the sink design water level. The minimum lake water level for the 100-year drought is elevation 682.3 feet, 5.3 feet higher than the minimum design operating level for normal station operation and 7.3 feet higher than the sink design water level. Low flow conditions in the creek do not influence the design water level and capacity of the ultimate heat sink due to its submerged condition. In the event of a failure of the main dam, or the occurrence of an extremely severe drought that would bring down the lake water level to less than elevation 677 feet, emergency shutdown operation will be initiated. In any event, the submerged ultimate heat sink will still be full of water and can supply cooling water for a safe shutdown operation.

The amount of sediment that can be deposited during the PMF has been conservatively calculated to be 262 acre-feet assuming that no sediment is deposited in the upper reaches of the lake and the entire sediment gets deposited in the UHS.

The capacity of the UHS will be sufficient for safe shutdown of the plant as shown in Table 9.2-12.

As described in Subsection 2.5.6.8, the UHS will be monitored for sediment accumulation periodically and after a major flood passes through the cooling lake. The accumulated sediment volume will be evaluated, and the UHS will be dredged prior to the accumulated volume reaching the available capacity for sediment accumulation of 218 acre feet.

A survey of the ultimate heat sink was done upon completion of construction in October 17, 1977. The topographical map of the completed UHS is shown in Figure 2.4-23. Beginning in November 1979, a yearly sounding survey of the bottom of the UHS was begun at 30 point locations as shown in Figure 2.4-31. In October 1981, the sounding survey was dropped in favor of the more accurate fathometer readings that provide a continuous depth measurement across the lake bottom. The locations for the sounding points and the fathometer readings are established through the use of survey markers around the UHS. Monitoring of the UHS shall be done on an annual basis for the first 5 years to determine the sediment deposition pattern. The subsequent monitoring interval would then be determined and adjusted on the basis of the results of the first 5 years. The monitoring program is described in Subsection 2.5.6.8.

A copy of 1980 Annual Sedimentation Survey is presented as Attachment A to this question. A summary of the procedures used in conducting the ultimate heat sink sedimentation monitoring program is also provided as Attachment B (Q&R 240.2).

(Q&R 240.2)

ATTACHMENT A

ULTIMATE HEAT SINK SEDIMENTATION MONITORING PROGRAM

Illinois Power Co.
Clinton Power Station
Clinton, Illinois

Prepared
by

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Decatur, Illinois
October 1980

(Q&R 240.2)

Ultimate Heat Sink Sedimentation Monitoring Program

Abstract:

Illinois Power Company has established a program to monitor the amount of sediment build-up on the bottom of the Ultimate Heat Sink (UHS) at the Clinton Power Station. This program calls the annual surveys to measure the sediment build-up. The first such survey was made during November 1979. A detailed outline of the procedures used in conducting this program has been outlined in report Number 1. The second annual survey was made during October 1980. Copies of the field notes and calculations for the October 1980 survey are included in the following pages of this report.

Summary of Findings:

Report No. 1 concluded that no significant measurable amount of silt build-up had occurred in the Ultimate Heat Sink from the date of impoundment to November 1979. The data from the October 1980 survey show that an average of approximately 0.2 feet of silt build-up has occurred between November 1979 and October 1980.

The average bottom elevation for the 30 survey locations being monitored was 667.7 feet for both the initial and November 1979 surveys. The average bottom elevation for the October 1980 survey was 667.9 feet. Based on a top of poolelevation of 675.0 feet for the UHS, this 0.2 foot difference constitutes a silt volume equal to 2.7% of the total initial volume of the UHS.

675.0' - 667.7' = 7.3' Ave. Depth

0.2' ÷ 7.3' = 2.7%

(Actual Field Survey Notes are not included in the USAR.)

(Q&R 240.2)

ATTACHMENT B

SUMMARY OF THE PROCEDURES
USED IN CONDUCTING THE
ULTIMATE HEAT SINK SEDIMENTATION
MONITORING PROGRAM

Illinois Power Co.
Clinton Power Station
Clinton, Illinois

ABSTRACT:

Illinois Power Company has established a program to monitor the amount of sediment build-up on the bottom of the Ultimate Heat Sink at the Clinton Power Station. This program calls for annual surveys to measure the sediment build-up. The first such survey was made during November 1979. A summary of the procedures used in conducting this program is given on the following pages of this report.

(Q&R 240.2)

OUTLINE OF FIELD & OFFICE PROCEDURES USED IN CONDUCTING THE SEDIMENTATION MONITORING PROGRAM FOR THE ULTIMATE HEAT SINK AT THE CLINTON POWER STATION.

Upon the completion of the final earthwork and grading for the Ultimate Heat Sink (UHS) and prior to the impoundment of the UHS, Illinois Power Company (IPC) had aerial photography taken of the UHS area. From this aerial photography, IPC had mapping prepared at a scale of 1"=100' with 2' contour intervals. From this aerial contour mapping, cross sections were plotted and the initial volume of the UHS below elevation 675.0' was calculated. The initial volume was calculated at 45,144,980 cu. ft. = 1036 acre-ft.

Permanent horizontal and vertical control points were established around the perimeter of the UHS. These horizontal and vertical control networks were established so as to be consistent with the aerial photography control system.

Thirty (30) locations were selected throughout the UHS basin at which periodic soundings are taken to determine the elevation of the top of the silt. The locations of these thirty sounding points and the location of the horizontal and vertical control points are shown on Figure 2.4-53. The initial UHS bottom elevations for each of these thirty locations were established from the aerial mapping.

The procedure used to conduct the UHS sediment monitoring program is as follows:

1. Theodolites are set up at two of the four horizontal control stations A, B, C, or D.

2. Each instrument man turns a predetermined angle so that the intersection of the lines of sight of the two theodolites falls at the exact location of the sounding to be taken.

3. By means of hand and/or radio communications between the two instrument men and the survey personnel in the boat, the boat is maneuvered to the correct location and the sounding is taken and recorded in the field book.

4. The locations of the two theodolites are changed as soundings are taken at various locations to avoid very narrow angles of intersections of the lines of sight.

5. A level circuit from the vertical control monuments to the UHS water surface is run at the beginning and end of each days soundings.

6. The top of silt elevations at each of the sounding locations is calculated by subtracting the sounding depth from the water surface elevation.

7. The difference between the initial UHS bottom elevation and the top of silt elevation as determined for the present survey is calculated and tabulated for each of the thirty monitoring positions.

8. The difference between the top of silt elevation as determined by the previous survey and the top of silt elevation as determined for the present survey is calculated and tabulated for each of the thirty monitoring positions.

(See Table 2.4-37 for sample data from #7 and #8 above)

9. Based on the data obtained in steps #7 and #8 above, a percentage of total silt build-up is calculated and an annual rate of silt deposition is calculated.

The surveys for the UHS settlement monitoring program as described in section 2.5.6.8 are scheduled to be done on a periodic basis with the further provision that additional surveys may be made as necessary to monitor the sediment build-up due to unusual conditions such as floods, earthquakes, etc.

The top of the roadway crossing North Fork upstream of the UHS is at elevation 703.7 feet. The length of the roadway that would be below probable maximum flood (PMF) elevation of 708.9 feet is 2,475 feet. The highway bridge consists of three spans of 65.83 feet, 78 feet, and 78 feet, respectively. The cross-sectional area of the bridge is 4,288 square feet. Low chord elevation is at 698.21 feet. The top of the roadway consists of 8-inch thick standard reinforced P.C.C. pavement. The sides of the embankment are protected with riprap. It is highly unlikely that the roadbed would be washed out during a major flood.

The length of the railroad embankment that would be under PMF level is 580 feet. The total volume of the fill material in the 580 feet long embankment is 67,900 cubic yards or about 42 acre-feet. The railroad is 1,600 feet upstream of UHS. It is very unlikely that the entire 580-foot long rail embankment would be washed out during one flood event. Even conservatively assuming that the entire fill material in the 580-foot long embankment would be washed out into UHS, the volume of UHS lost is only 42 acre-feet. This volume is small compared to the total volume of UHS of 1,067 acre-feet (Q&R 240.3).

2.4.12 <u>Dispersion, Dilution, and Travel Times of Accidental Releases of Liquid Effluents in Surface Water</u>

The effects of accidental releases of liquid effluents in surface waters are evaluated for components containing liquid radioactive materials located outside the containment building. The radwaste tanks with the highest total radioactive inventory are the phase separator tanks. The two phase separator tanks have a capacity of 10,000 gallons per tank.

The tanks are located in separate concrete cells in the basement of the radwaste building at elevation 702 feet. The ambient groundwater level at the plant is 730 feet and the plant grade elevation is 736 feet. Therefore, the only way any effluents released accidentally through postulated cracks in the radwaste building can reach a surface water body is by entering the surrounding groundwater environment.

As discussed in Subsection 2.4.13.3, there is no possibility that the effluents can move out of the building due to high groundwater elevation. Therefore, it is not possible for the effluents to reach a surface water body. Furthermore, the portion of the radwaste building located below grade is designed as a seismic Class I structure to ensure integrity.

The locations of surface water users are discussed in Subsection 2.4.1.2. There are no known surface water users of Salt Creek or Sangamon River within 50 river miles downstream from the plant site. The closest surface water user for drinking purposes is inAlton, Illinois on the Mississippi River, 242 river miles downstream from the CPS site.

2.4.13 Groundwater

Physical characteristics, yield, and groundwater quality of the major aquifers are discussed in this subsection. The site work, particularly the excavations for the main plant (power block), ultimate heat sink, and main dam, and the installation of the CPS test well, has confirmed that site hydrogeologic conditions are as anticipated from the PSAR-stage investigations with the exception of the quality of groundwater in the buried Mahomet Bedrock Valley. As a result of the high methane content of the groundwater obtained from the test well, the potable and sanitary water requirements for the Clinton Power Station are met from Lake Clinton rather than from groundwater wells as stated in the PSAR.

2.4.13.1 Description and Onsite Use

2.4.13.1.1 Onsite Use

Groundwater is not used for the Clinton Power Station during plant operation. As stated in the PSAR, potable and sanitary water requirements were to have been supplied by groundwater from sand and gravel deposits in the buried Mahomet Bedrock Valley. However, an alternate source (Lake Clinton) was chosen when water with a high methane content was obtained from a test well drilled approximately 1 mile south of the plant site (Figure 2.4-32). Physical characteristics of the CPS test well are given in Table 2.4-27. The quality of groundwater from this well is summarized in Table 2.4-28.

A gas flow measurement made during test pumping of the CPS test well revealed the presence of burnable gas in the water samples. Results of two gas analyses indicated that methane comprised more than 80% of the total gas sample. Methane is commonly reported in water wells finished in the glacial drift in central Illinois and is thought to be produced during decomposition of organic materials contained within the drift (Reference 45). The volume of gas in water samples from the test well is similar to that reported for other gas-producing water wells in DeWitt County (Reference 46).

Water requirements during plant construction were supplied from surface water sources.

2.4.13.1.2 Regional Hydrogeologic Systems

Hydrogeologic systems within 15 miles of the Clinton Power Station include the following aquifers and aquitards:

a. alluvial deposits along present streams;

b. glacial drift, including layers and lenses of sand and gravel within and between the various tills;

c. glacial outwash in buried bedrock valleys;

d. bedrock of Pennsylvanian age, consisting of shale, siltstone, limestone, sandstone, underclay, and coal;

e. bedrock of Silurian, Devonian, and Mississippian age, predominantly dolomites and limestones; and,

f. the Cambrian-Ordovician aquifer system, a sequence of limestones, dolomites, and sandstones.

Groundwater supplies are obtained chiefly from the glacial outwash in the buried bedrock valleys and shallower unconsolidated deposits and, to a minor extent, from the upper 100 feet of the Pennsylvanian rock sequence beneath the glacial drift. The lower bedrock aquifers are not used for water supply in this area since adequate supplies for municipal, agricultural, and domestic requirements are more easily obtained from the shallower bedrock or the overlying unconsolidated materials. Moreover, poor quality water in the deeper aquifers is typical in this region. For these reasons, the hydrogeologic characteristics of the stratigraphic units below the Pennsylvanian bedrock are not discussed in the following subsections. Hydrogeologic characteristics of the glacial deposits and the Pennsylvanian bedrock are summarized in Table 2.4-29.

Alluvial deposits, consisting of varying amounts of clay, silt, sand, and gravel, occur in the valleys of many streams in the regional area. The alluvium may be used for groundwater supply in those areas where thick, permeable sand and gravel deposits are present. Such deposits commonly occur along larger streams having established flood plains, such as Salt Creek and North Fork of Salt Creek. Alluvial aquifers are not used extensively in the regional area because the flood plain areas have undergone only minor development. The public water supply for Heyworth, in McLean County, is obtained from alluvial deposits along Kickapoo Creek. Pumping tests showed the aquifer at this location to be capable of supplying over 200 gallons per minute (gpm) per well (Reference 47). The following concentrations were reported for selected chemical constituents in groundwater from the alluvial aquifer at Heyworth for a sample collected in 1972: hardness (as $CaCO_3$ 284 parts per million (ppm); alkalinity (as $CaCO_3$), 240 ppm; chloride, 16 ppm; total iron, 0.4 ppm; and total dissolved minerals, 329 ppm (Reference 47).

With the exception of surficial alluvium in present stream valleys, the regional area is underlain by a thick sequence of silts of eolian and lacustrine origin, tills, and outwash, collectively known as glacial drift. The total thickness of these deposits varies from less than 50 feet to approximately 400 feet, and averages 200 feet (Reference 48). The silts are often clayey and may contain fine sand. The tills are composed of heterogeneous mixtures of clay, silts, sand, and gravel but consist predominantly of clayey silts or silty clays; lenses and thin, discontinuous layers of silt, sand, or gravel are common between and within the tills. Outwash deposits consist of sand and gravel with varying amounts of silt or clay. Detailed descriptions of the various unconsolidated deposits and their stratigraphic relationships are presented in Subsection 2.5.1.2.2.1.

Availability of groundwater from the unconsolidated deposits is governed by the occurrence of permeable sand and gravel aquifers within the glacial drift. Sand and gravel deposits may occur above or below the individual tills, as lenses within the tills, or as relatively continuous deposits in bedrock valleys (Reference 48). The Wisconsinan-age formations are generally composed of fine-grained sediments with only shallow and very localized deposits of sand and gravel and, as such, are poor sources of groundwater. Widespread lenses of sand and gravel intercalated in the Illinoian drift are capable of supplying small to moderate amounts of groundwater (Reference 48). Sand and gravel deposits in the Kansan drift occur primarily as outwash deposits in buried bedrock valleys; the axes of the bedrock valleys are shown in Figure 2.5-7. Groundwater in the Kansan and Illinoian deposits occurs under artesian conditions whereas in the Wisconsinan deposits water table conditions generally prevail (Table 2.4-29).

Within 15 miles of the station site, the most productive aquifer consists of Kansan outwash deposits (Mahomet Sand Member) in the buried Mahomet Bedrock Valley and its tributaries. Wells near the margins of the bedrock valleys may produce as much as 500 gpm. Wells located in the center of the valleys might yield substantially higher quantities of groundwater on a sustained basis given proper well construction and management (Reference 49). Most wells in this area do not produce from this zone, however, because adequate supplies for domestic, agricultural, and most municipal purposes may be developed from the alluvium along stream courses or from small permeable lenses in the upper glacial drift materials.

Groundwater in the glacial drift is derived from precipitation, underflow through bedrock and bedrock valleys, and induced infiltration from streambeds (Reference 48). Recharge to the sand and gravel deposits occurs primarily by vertical leakage of infiltrating precipitation, the rate of which is controlled by the vertical permeability of the relatively impermeable tills, the thickness of the tills (confining beds), and the head differential between the source of recharge and the receiving aquifer (Reference 50). Walton (Reference 50) reported that vertical permeability for till with some sand and gravel averages 0.02 gpd/ft^2 (1.0 x 10^{-6} cm/sec). The recharge rate for sand andgravel aquifers overlain by thick glacial drift consisting largely of till is 115,000 gpd/mi^2 (Reference 50).

Groundwater in the glacial drift aquifers is discharged to streams that intersect the aquifers (groundwater runoff), to the underlying glacial drift, to the Pennsylvanian bedrock, and to pumping wells. Groundwater runoff for the upper portion of the Salt Creek drainage basin, calculated from hydrologic data collected at the Rowell gauging station, averages 0.36 cfs/mi^2 for years of near-normal precipitation. Groundwater runoff averages 0.13 cfs/mi^2 for years of below-normal precipitation and 0.58 cfs/mi^2 for years of above-normal precipitation (Reference 50). Bank storage in alluvial deposits accounts for much of the variability in observed values of groundwater runoff between years of below-normal and above-normal precipitation.

Groundwater quality in the Illinoian and Kansan aquifers is summarized in Table 2.4-30. As indicated in the table, the quality of groundwater does not differ substantially between aquifers (Reference 51). Water from wells tapping Wisconsinan aquifers generally has a lower mineral content than water from wells in the deeper formations. However, the quality of groundwater obtained from Wisconsinan aquifers is more variable, which in part is due to local contamination of shallow wells from nearby pollution sources such as septic tanks and feedlots (Reference 52). Iron content in water from the deeper wells almost always exceeds the recommended upper limit of 0.3 mg/l set by the U.S. Public Health Service. The high chloride content reported for some wells in the Illinoian and Kansan aquifers suggests that some highly mineralized water is being discharged from the Pennsylvanian bedrock to the overlying glacial deposits in some areas (Reference 52). In addition to the CPS test well, methane gas is present in seven public water supply systems within 15 miles of the site. Methane is also reported from numerous private wells in the regional area.

The glacial drift is underlain by Pennsylvanian bedrock that consists largely of shale and siltstone interbedded with limestone, sandstone, underclay, and coal. The Pennsylvanian strata are described in greater detail in Subsection 2.5.1.2.2.2. Small amounts of groundwater may be obtained from wells penetrating beds of sandstone, creviced limestone, and fractured shale and coal. Recharge to the Pennsylvanian bedrock occurs by vertical leakage from the overlying glacial drift. Groundwater in the bedrock is under artesian conditions and is discharged to lower bedrock formations or to the glacial drift in those areas where the potentiometric surface of the Pennsylvanian aquifers is higher than that of the drift aquifers. Most bedrock wells extend less than 100 feet below the bedrock surface because the formations become more tight and mineralization of the groundwater increases with depth (Reference 53). Bedrock is used as a source of domestic water supply in the regional area only where conditions are unfavorable for the development of drift aquifers.

2.4.13.1.3 Site Hydrogeologic Systems

The hydrogeologic systems in the site area consist of alluvial deposits along Salt Creek and North Fork of Salt Creek, glacial drift, glacial outwash in the buried Mahomet Bedrock Valley, and Pennsylvanian-age bedrock. The occurrence of aquifers in each hydrogeologic system is described in Subsection 2.4.13.1.2 along with the general characteristics of yield, recharge and discharge, and quality of groundwater for the aquifers. The data presented in this subsection are based upon site investigations.

Alluvial deposits (Henry Formation) in the vicinity of the ultimate heat sink for Clinton Power Station consist of finegrained flood plain deposits overlying coarse-grained outwash. Illinoian till (Glasford Formation) underlies all alluvial deposits. The flood plain deposits are commonly silt with some fine sand and clay, whereas the outwash deposits are sand and gravel with varying amounts of silt or clay. The total thickness of the alluvial deposits varies from 6 to 48 feet in the ultimate heat sink borings and averages 18.5 feet. Flood plain deposits range from zero to 23.2 feet thick and average 9.1 feet thick. Outwash deposits range from zero to 41 feet thick and average 9.2 feet thick; the thickest outwash deposits are located over an apparent terrace on the north side of the valley. Outwash deposits were observed to be continuous in the foundation excavation for the ultimate heat sink dam. The base of the outwash in the borings ranges in elevation from 678.3 to 650.5 feet with the most frequently reported base elevations in the interval between 667 and 657 feet. Permeability tests were not performed in the ultimate heat sink borings. However, based upon the results of particle-size analyses for samples from the PH and D-series borings (Figures 2.4-33 and 2.4-34), the permeability of the outwash deposits is on the order of 10^{-3} to 10^{-2} cm/sec. There were no known domestic or farm supply wells in the alluvial deposits in the ultimate heat sink area.

The sequence of glacial drift exposed in the CPS power block excavation consists of the Wisconsinan-age Richland Loess, Wedron Formation, and Robein Silt, and the Illinoian-age Glasford Formation. The lithologies of these stratigraphic units are summarized in Table 2.4-29 and are described in more detail in Subsection 2.5.1.2.2.1. Fifteen deep borings in the plant and ultimate heat sink areas encountered lacustrine deposits and Kansan-age till beneath the Illinoian-age drift (Subsection 2.5.1.2.2.1). The total thickness of the glacial drift in the plant area varies from 229.9 to 250.3 feet and averages 237.4 feet.

Several sand lenses within the till were penetrated by the station site borings. Most of the lenses occur between elevations 650 and 730 feet and range in thickness from several inches to 22 feet. The excavation penetrated some of these lenses, while others lie within 25 feet of the base of the excavation (elevation 680 feet). Figure 2.4-35 shows the distribution of sand lenses in the site vicinity. The total areal extent of these features is unknown.

The interpretation of the sand deposits encountered in the borings as discontinuous pockets or lenses was generally confirmed in the geologic mapping of the power block excavation (Attachment C2.5). The one exception is a nearly continuous layer of fine sand observed in the excavation near the top of the Wedron Formation. In addition, sand is reported at the same position in most of the borings around the plant except those within the triangular area formed by the ultimate heat sink baffle dike abutment, the screen house, and the southwest corner of the excavation. In general, the base of the sand layer slopes from elevation 723 feet at the western limit of the excavation to elevation 716 feet on the slope above the cooling lake. In borings between the excavation and the cooling lake, the thickness of the sand layer varies from 2.0 to 16.5 feet. The remainder of the sand deposits exposed in the excavation occurred as discontinuous seams and localized pockets within the tills of the Wedron and Glasford Formations. Sand pockets also occurred near the contact between the weathered and unweathered portions of the Glasford Formation. The combined quantities of precipitation and groundwater seepage into the excavation from the sand deposits was controlled using ditches and sumps.

Falling-head and constant-head type permeability tests were performed in the laboratory on representative soil samples of the Henry Formation (Salt Creek alluvium), and the Glasford Formation (Illinoian glacial till and interglacial zone). The tests were performed in the manner described in Figure 2.5-347, which resulted in measurements of the vertical permeability of each soil formation. The results of these tests are presented in Tables 2.5-31 and 2.5-32. Only one sample of the Henry Formation was tested, the results of which indicate a vertical permeability of 1.8×10^{-8} cm/sec for the fine-grained flood plain deposits; the underlying outwash was not tested. Vertical permeability of sand samples from the weathered portion of the Glasford Formation averages 2.1×10^{-3} cm/sec, ranging from 1.8×10^{-4} to 4.7×10^{-3} cm/sec. In the unaltered Glasford Formation, the vertical permeability ranges from 3.8×10^{-9} to 2.3×10^{-7} cm/sec, and averages 3.8×10^{-8} cm/sec. Also presented in Table 2.5-32 is an estimate of the porosity for each sample. The porosity was calculated using laboratory data which included degree of saturation, wet density, moisture content, and an assumed specific gravity. The formulae used are present in any standard textbook on soil mechanics.

Falling-head type field permeability tests were performed at the Lake Clinton dam site and the station site. The tests were performed in piezometers installed to the specified zone of percolation in the borehole. The tube connecting the piezometer was first filled to the top with water and the time required for the water level to drop a given distance was recorded. Each test was performed several times until consistent results were obtained. Permeability values as calculated from these tests represent the average horizontal permeability within the zone of percolation in the borehole and are shown in Table 2.5-38. Average horizontal permeability values range from 1.2×10^{-6} to 2.6×10^{-6} cm/sec in the Wisconsinan till and 6.1×10^{-6} to 1.4×10^{-5} cm/sec in the Illinoian till. The field tests indicate relatively higher permeabilities in the Illinoian till than the laboratory test results, and probably represent more nearly the actual rate of groundwater movement through the in situ soils.

The configuration of the water table in the immediate vicinity of the station site was measured by means of piezometers installed in borings during initial site investigations. The pertinent piezometer data are shown in Table 2.4-31. The groundwater measurements are shown graphically in Figures 2.4-36 through 2.4-43 and 2.4-48 through 2.4-50. Groundwater exists under water table conditions in the Wisconsinan till and under confinement in the underlying Illinoian and Kansan tills. Piezometer levels measured at the station site averaged about elevation 713 feet in the Illinoian till and approximately elevation 680 feet in the Kansan till over the period of observation. The potentiometric level in the Kansan outwash deposits of the buried Mahomet Bedrock Valley, as measured in the CPS test well, was approximately elevation 600 feet MSL.

Recharge to the aquifers in the glacial drift and upper bedrock is by vertical leakage from the overlying fine-grained deposits (Subsection 2.4.13.2.3) and by underflow moving down-gradient toward the site vicinity from the east and northeast. Similarly, groundwater in these aquifers is discharged to underlying aquifers. Some groundwater in the upper glacial drift deposits discharges from springs present within the general vicinity of the proposed power station and cooling lake area. A survey was conducted by use of airphoto interpretations, field reconnaissance, and personal interviews with local farmers. The springs found during this survey are shown in Figure 2.4-44. None of these springs are currently being used as a potable water supply.

Chemical analyses of groundwater samples from selected borings in the glacial deposits were made and are shown in Table 2.4-32.

2.4.13.2 Sources

2.4.13.2.1 Present and Future Groundwater Use

Groundwater usage is discussed regionally (within 15 miles from the site) only with respect to non-private wells. Within 5 miles of the site, both public and private wells are reviewed.

Public water supplies in the regional area are derived exclusively from groundwater sources. Public wells within 15 miles of the site, inventoried during the PSAR-stage investigations, are listed in Table 2.4-33 and are shown in Figure 2.4-45. There are no known industrial wells in this area. Private wells beyond 5 miles have not been tabulated. Public water supply systems within 15 miles of the site are summarized in Table 2.4-34. The number of producing wells, the producing hydrogeologic units or aquifers, and the average daily usage are identified for each public water supply system.

Future water supplies for the village of DeWitt will be provided by a small test well drilled by Illinois Power Company in 1974 (Tables 2.4-33 and 2.4-34). Distribution piping from this well, located about 120 feet southeast of the CPS test well was installed during September 1979.

Within 15 miles of the station site, approximately 65% of the total public groundwater supplies is pumped from the Mahomet Bedrock Valley aquifer. Except for the alluvial wells at Heyworth, the remaining public water supplies are pumped from wells in the Wisconsinan, Illinoian, and Kansan glacial deposits. Bedrock wells are not used in any public water supply system.

There are approximately 141 private wells for domestic and stock watering use within 5 miles of the site as determined from published records and a field inventory conducted during the PSAR-stage investigations (Table 2.4-35 and Figure 2.4-46). The records for many of these wells were filed over 25 years ago and a substantial number may not be in use at the present time. Non-private use of groundwater within 5 miles of the site consists of two school supplies (Table 2.4-35, well numbers 12 and 120). No industrial wells are known to exist in this area.

Of the domestic wells listed in Table 2.4-35, 37 are located on the site property. In addition, Illinois Power Company located 73 onsite large diameter water wells not listed in Table 2.4-35. Records are not available for these large diameter wells. Most of the domestic wells are less than 150 feet deep and produce from sand lenses in the upper glacial tills rather than from the deeper Mahomet Bedrock Valley aquifer. Production exceeded 10 gpm in only a few cases. With the exception of wells used by tenant farmers or for monitoring (Subsection 2.4.13.4), wells on the site property were abandoned and sealed in accordance with applicable state requirements during plant construction.

There is one private residence on a small peninsula situated one and one half miles southwest of the power block. The peninsula is bordered on the north, west and south by Lake Clinton.

The water supply for the residence is provided by three drilled wells. One well is 80 feet deep and is installed in Illinoian till. The other two wells are 35 feet deep and are estimated to be completed near the top of unaltered Illinoian till.

A second residence is located on site property three quarters of a mile southwest of the power block. This farmstead is served by three wells. One well is 247 feet deep and is installed in the buried Mahomet bedrock valley aquifer, which is not present beneath the station site (Subsection 2.4.13.2.2). The other two wells were dug 30 feet deep and are estimated to be completed near the top of unaltered Illinoian till.

The wells for both residences are expected to remain in use during plant operation. The locations and available construction details for the six wells have been added to Figure 2.4-46 and Table 2.4-35. Based on the distances of the wells from the power block the topography and the hydrogeologic conditions presented in Subsection 2.4.13, the potential for contamination of the wells due to plant operation is minimal. It should be noted that the radioactive liquid stored in tanks in the Radwaste Building will not leak out of the building (refer to the response to Question 240.8), and therefore the groundwater regime will not be contaminated (Q&R 240.10).

The area within 15 miles of the site includes most of DeWitt County and portions of Macon, McLean, and Piatt Counties. Available groundwater supplies for DeWitt County exceed 39 mgd (Reference 54). Public groundwater use from Table 2.4-34 totals 1.42 mgd in DeWitt County. Rural groundwater use in the county is estimated from Reference 54 to be approximately 0.4 mgd, making the present water demands less than 2 mgd, or approximately 5% of the total available supplies. Thus, groundwater is capable of meeting any foreseeable increase in water demand in DeWitt County. Similar conclusions may be made for the rest of the regional area since the hydrogeologic and population characteristics of the other counties are similar to those for DeWitt County.

2.4.13.2.2 Regional Hydrogeologic Conditions

The water table in the upper (Wisconsinan) glacial deposits generally occurs within a few feet of the ground surface. Groundwater levels are deepest over togographically high areas and shallowest in topographically low or flat areas. Groundwater levels have been measured regionally by the Illinois State Water Survey in a statewide network of observation wells. The water table in wells finished in Wisconsinan deposits varies from 2 to 19 feet below the ground surface. Seasonal fluctuations in individual observation wells range from 1.5 to 12 feet and average approximately 5 feet (Reference 51). Water levels are highest during spring when conditions are most favorable for recharge from precipitation. The water table falls from the spring peak during late spring, summer, and early fall when discharge by evapotranspiration and groundwater runoff exceeds recharge from precipitation (Reference 51). Regional groundwater movement on the Wisconsinan till plain is generally west and southwest toward the Illinois River under a hydraulic gradient of approximately 2 or 3 feet per mile. The water table is locally deflected and steepened toward stream courses that cross the till plain and are tributary to the Illinois River.

Reversals in the regional hydraulic gradient and regional declines in the potentiometric surface have resulted from intensive pumping in the heavily urbanized Champaign-Urbana district 32 miles to the east, where pumpage is from the Mahomet Bedrock Valley aquifer (Reference 51). No positive evidence of these effects has been identified in DeWitt County to date, but declines on the order of 1 or 2 feet may eventually occur in the eastern portion of the county if pumpage is doubled to approximately 30 million gallons per day in the Champaign district by the year

2000 (Reference 51). These declines probably will not be significant at the site and no changes in the local pattern of groundwater movement would be expected to occur.

Closer to the site, reversals in the hydraulic gradient may also be expected to occur in response to pumping from the City of Clinton municipal well field. Lower potentiometric levels within the cone of influence induce higher recharge rates to the Mahomet Bedrock Valley aquifer which may, in turn, cause potentiometric levels in the overlying aquifers to decline slightly within the cone of influence. However, the cone of influence associated with the City of Clinton municipal well field is much smaller than the cone developed around Champaign-Urbana because pumpage at Clinton (Table 2.4-33) totals less than one-tenth of that at Champaign Urbana. The cone of influence at Clinton is probably limited to an area within a few miles of the well field and should have little, if any, effect on groundwater levels at the station site. In addition, the main plant borings showed the buried Mahomet Bedrock Valley is not present beneath the station site.

2.4.13.2.3 Site Hydrogeologic Conditions

Piezometers were installed in selected borings during 1972 and 1973 to establish the configuration of the water table in the immediate vicinity of the site. Additional piezometers were installed around the lake during construction in 1976 (OW-1 through OW-8) and downstream from the dam in 1977 and 1979 (OW-9 through OW-24). Some of the piezometers are no longer functional, having been destroyed by construction activities. A summary of the installation dates, tested intervals, and status of the piezometers is presented in Table 2.4-31. Locations of piezometers other than the P-series (Main plant) and OW-9 through OW-17 (below dam) are shown in Figure 2.4-32. Locations of piezometers installed in P-series borings are shown in Figure 2.5-16; locations of OW-9 through OW-24 are shown in Figure 2.5-272. Installation details for the piezometers are given on their respective boring logs. A typical OW-series piezometer is shown in Figure 2.4-47. Groundwater levels measured in the piezometers through 1979 are shown graphically in Figures 2.4-36 through 2.4-43 and 2.4-48 through 2.4-50.

In the plant area, the highest groundwater level was measured in the Wisconsinan till at elevation 729.7 feet MSL in the piezometer installed in boring P-40 (Table 2.4-36). Groundwater levels in seven piezometers installed in Illinoian deposits ranged in elevation from 675 to 717 feet MSL. In the two piezometers installed in Kansan deposits, the potentiometric level coincided with the base of the main plant excavation at elevation 680 feet MSL. Despite the high groundwater levels, seepage into the excavation was minor and was controlled using a system of drainage ditches and sumps. The low volume of seepage was due to the low overall permeability of the tills (Subsection 2.4.13.1.3) and the limited areal extent of the discontinuous sand layers and lenses within the tills.

From the onsite borings, the top of the Illinoian deposits averaged 698 feet MSL and the top of the Kansan deposits averaged 572 feet MSL. Comparison of these elevations with the potentiometric levels measured in the Wisconsinan, Illinoian, and Kansan deposits indicates that groundwater occurs under water table conditions in the Wisconsinan deposits and under confined conditions in the underlying Illinoian and Kansan deposits. The head relationships between the Wisconsinan, Illinoian, and Kansan aquifers also indicate that the glacial drift aquifers are recharged by vertical seepage from the overlying drift under a net downward hydraulic gradient.

The water table in the vicinity of the station site occurs as a ridge-like mound in the Wisconsinan till between Salt Creek and North Fork of Salt Creek (Figure 2.4-32). The position of the groundwater ridge marks a recharge area from which groundwater flows to the southeast toward Salt Creek and to the northwest, across the plant site, toward North Fork of Salt Creek. The magnitude of the hydraulic gradient at the plant site is approximately 0.086, or 454 ft/mi. This value is based upon a maximum head loss of 55 feet over a minimum distance of 640 feet from the plant site to the edge of the flood plain of North Fork of Salt Creek.

Prior to impoundment of the cooling lake, North Fork of Salt Creek served as the local base level for groundwater flow from the plant area to the flood plain. Impoundment of the cooling lake has raised the base level to elevation 690 feet MSL, causing the groundwater-surface water interface to shift to the southeast toward the plant. Assuming no change in the height of the water table at the station, establishment of a new base level higher and closer to the station will eventually result in a decrease in the hydraulic gradient to about 0.068, or 359 ft/mi.

Filling of the cooling lake began on October 12, 1977, and normal pool level was attained on May 17, 1978. As shown by piezometer measurements (Figures 2.4-39 through 2.4-43 and 2.4-48 through 2.4-50), lake filling has not caused substantial readjustment of groundwater levels upstream from the dam. The largest increases in groundwater levels were observed in the vicinity of the dam abutments in piezometers screened in the Illinoian deposits. This response was anticipated because the base of the overlying Wisconsinan deposits in the onsite borings is approximately eight feet above the normal pool elevation of 690 feet MSL. In 1978 and 1979 high hydrostatic pressures were observed in the Illinoian deposits beneath the floodplain downstream from the east dam abutment. Five relief wells (three presently being used) installed in October 1979 have reduced the excess pressures, as shown on Figures 2.4-49 and 2.4-50.

2.4.13.3 Accident Effects

As described in Subsection 2.4.12, the tanks which are located outside the containment building and contain the highest total radioactive inventory, are the phase separator tanks. Each of these tanks has a capacity of 10,000 gallons with the floor elevation at 702 feet. The design groundwater elevation at the plant site is 730 feet. The plant grade elevation is 736 feet.

To examine the impact of a postulated accidental release of radioactive effluents, it is hypothesized that one of the phase separator tanks spills its contents into the concrete cell in which it is located. The walls and 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 707 feet. The level of radioactive effluents in the building would have to exceed the groundwater level at the plant before seepage out of the building could occur. As the ambient groundwater elevation is 23 feet higher than the fluid level inside the cell, there would be no hydraulic gradient from the interior of the cell to the outside*. Rather, groundwater would be forced into the building to relieve the prevailing hydrostatic pressure. Therefore, it is extremely unlikely that any radioactive effluent would be released from the building to the surrounding groundwater environment.

* The statement cited refers to the ambient groundwater condition when the main plant area is completely backfilled to the final grade elevation. In case of an accidental spill, contaminants would drain towards the sumps in the floor of the radwaste building and would be pumped out for further treatment by the radwaste treatment systems. All the openings that lead into Unit 2 excavation

have been closed and waterproofed. Hence, contaminants would not be discharged into the excavation due to accidental spill (Q&R 240.8 and 240.9).

The only waste storage tanks located above grade are the Concentrate Waste Tanks which are located at elevation 762 feet in the radwaste building. In case of a postulated rupture of one of these tanks, the contents would ultimately reach the basement of the building through the floor drain system, and would also be contained there due to high ambient groundwater level.

2.4.13.4 Monitoring

Groundwater levels in the vicinity of the cooling lake and power block have been monitored intermittently since site investigations began in 1972. The present groundwater monitoring system is described in Subsection 2.4.13.2.3. Piezometers OW-1 through OW-8 were installed in August 1976 to monitor the effect of the cooling lake on surrounding groundwater levels. Piezometers OW-9 through OW-17 were installed downstream from the dam in August 1977 to monitor dam performance. Additional piezometers, OW-18 through OW-24, were installed downstream of the dam in August and October 1979. Future piezometers will be installed in the vicinity of the power block before the continuation of construction of Unit 2 or the backfilling of the excavation of Unit 2 commences. These piezometers will be used to verify the groundwater level used for the main plant hydrostatic loading. Monitoring of groundwater levels around the cooling lake will continue for a minimum of 5 years after filling of the lake, or until equilibrium between lake levels and groundwater levels is established. Monitoring of piezometers in the vicinity of the dam is addressed in the monitoring program for dam performance.

IP began preoperational monitoring of several water wells around the periphery of the cooling lake in 1978, as required by the NRC Environmental Statement. The purpose of this monitoring, conducted semiannually, is to identify any changes in groundwater quality, either chemical or biological, resulting from the impoundment of Lake Clinton. IP sampled a total of nine public and private wells in 1978 and added well numbers 89 and 126 (Table 2.4-35 and Figure 2.4-46) to the sampling program in 1979.

An operational groundwater monitoring program was implemented to assure early detection of groundwater contamination resulting from either normal plant operation or an accidental effluent release. There are no groundwater users downgradient from the plant (between the power block and the cooling lake) and rapid groundwater movement through the discontinuous sand deposits within the glacial tills is precluded by the relative impermeability of these tills. However, since some areas around the cooling lake have been designated for use by the general public, a monitoring program was established one year prior to fuel loading to provide background chemical and radiological data.

The preoperation and early operational monitoring program for radiological monitoring of wells covered only two wells as discussed in the Environmental Report - Operating License Stage Subsection 6.1.5 and Table 6.1-2. The two wells are the DeWitt water supply well and a well drilled in the recreation area. Additional wells were added in the 2006 timeframe to provide a comprehensive groundwater sampling and analysis program as part of the Radiological Groundwater Protection Program. The monitoring programs include gamma isotopic analyses performed on groundwater samples collected from the wells listed above. Details of the preoperational and operational groundwater monitoring programs are described in the Environmental Technical Specifications for the Operating License.

2.4.13.5 Design Basis For Hydrostatic Loading

The groundwater level assumed for calculation of hydrostatic loading on the power plant foundation is elevation 730 feet MSL.

Power block foundations are below the groundwater levels measured in the Wisconsinan deposits during site investigations (Subsection 2.4.13.2.3). Precipitation and groundwater seepage into the excavation were controlled by means of ditches and sumps.

2.4.14 Technical Specifications and Emergency Operation

The nuclear station, together with its associated safety-related facilities, is designed to function and shutdown in a safe manner despite the occurrence of any of the adverse hydrological events discussed in the preceding sections. Therefore, technical specifications outlining emergency procedures for plant shutdown are not deemed necessary.

A discussion of the effects which several adverse hydrological events will have on the safety-related facilities is included as a justification for the exclusion of emergency procedures.

2.4.14.1 Flooding

The probable maximum flood level as discussed in Subsection 2.4.3.5 is 708.9 feet. The station grade is 736 feet, and therefore flooding cannot affect the safety of the station.

The station site drainage system is designed to carry away probable maximum precipitation, and access to the site is not limited by severe hydrological events.

The Seismic Category I circulating water screen house is designed to behave elastically and remain functional and watertight under the effects of water levels, including PMF and the associated wind-generated waves.

2.4.14.2 Low Water Level

The station ultimate heat sink is submerged in the cooling lake, so that low flow conditions have no influence on the shutdown service water source. And as further discussed in Subsection 9.2.5, these drought conditions will not affect station safety since the reactor must be placed in safe shutdown due to lack of circulating water source.

Sedimentation of and its effect on the ultimate heat sink is discussed in Subsection 2.4.11.6.

2.4.15 References

1. W. D. Mitchell, "Floods in Illinois: Magnitude and Frequency," Illinois Division of Waterways and U.S. Geological Survey, 1954.

2. W. D. Mitchell, "Unit Hydrographs in Illinois," Illinois Division of Waterways and U.S. Geological Survey, 1948.

3. U.S. Geological Survey, "Compilation of Records of Surface Waters of the United States through September 1950," Part 5, Water Supply Paper 1308, Washington, D.C., 1959.

4. U.S. Geological Survey, "Compilation of Records of Surface Waters of the United States, October 1950 to September 1960," Part 5, Water Supply Paper 1728, Washington, D.C., 1964.

5. U.S. Geological Survey, "Water Resources Data for Illinois 1960 to 1976."

6. W. J. Roberts and J. B. Stall, "Lake Evaporation in Illinois," Report of Investigation 57, Illinois State Water Survey, 1967.

7. W. L. Denmark, "The Climate of Illinois," Climates of the States, (by officials of NOAA, U.S. Dept. of Commerce) pp. 90-107, Water Information Center, Port Washington, N. Y., 1974.

8. D. W. Oehmke, Sanitarian, Division of Public Water Supplies Illinois EPA, Personal Communication, 1973.

9. R. L. Wheatley, Assistant Chief, Division of General Sanitation, Illinois Department of Public Health, Personal Communication, 1973.

10. J. P. Anderson, Division Manager, Division of Public Water Supplies, Illinois EPA, Personal Communication, 1972.

11. G. E. Gehrke, Logan County ASCS Executive Director, Personal Communication, 1973.

12. A. D. Koberners, Cass County ASCS Executive Director, Personal Communication, 1973.

13. O. Sprague, DeWitt County ASCS Executive Director, Personal Communication, 1973.

14. D. W. Leithoff, Mason County ASCS Executive Director, Personal Communication, 1973.

15. U.S. Corps of Engineers, "Storm Rainfall in the United States, Depth-Area-Duration Data," Washington, D.C., 1946 (with later supplements).

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

17. U.S. Army Corps of Engineers, "Standard Project Flood Determinations," EM 1110-2-1411, March 1952, Revised March 1965.

18. V. T. Chow, "Handbook of Applied Hydrology," pp. 14-15 to 14-24, McGraw-Hill Book Co., New York, 1964.

19. D. M. Hershfield, "Rainfall Frequency Atlas of the United States," Technical Paper No. 40, U.S. Weather Bureau, May 1961.

20. J. H. Davis and M. L. Terstriep, "Potential Surface Water Reservoirs of North-Central Illinois," Report of Investigation No. 56, Illinois State Water Survey, Urbana, Illinois, 1966.

21. University of Illinois Agricultural Experiment Station, "DeWitt County Soils," Soil Report No. 67, June 1960.

22. University of Illinois Agricultural Experiment Station, "Soils of Illinois," Bulletin 725, August 1967.

23. U.S. Army Corps of Engineers, Written Communication on SPF and PMF for the Illinois River on November 2, 1970.

24. U.S. Army Corps of Engineers, "Spillway Rating and Flood Routing," Computer Program 22-J2-L 210, Hydrologic Engineering Center, October 1966.

25. U.S. Army Corps of Engineers, "Water Surface Profiles," Computer Program 22-J2-L 232, Hydrologic Engineering Center, December 1968.

26. V. T. Chow, "Open Channel Hydraulics," McGraw-Hill Book Co., New York, 1959.

27. U.S. Army Corps of Engineers, "Shore Protection, Planning and Design," Technical Report No. 4, 3rd Edition, Coastal Engineering Research Center, 1966.

28. U.S. Army Corps of Engineers, "Waves in Inland Reservoirs," Technical Memorandum No. 132, November 1962.

29. Surface Water Supply of the United States, 1959, Part 5, Rating Table, Water Year 1958-59, p. 492.

30. J. B. Stall, "Low Flows of Illinois Streams for Impounding Reservoir Design," Bulletin 51, Illinois State Water Survey, 1964.

31. J. B. Stall, N. L. Rupain, and P. K. Kandaswamy, "Sediment Transport in Money Creek," Journal of the Hydraulics Division, ASCE, Vol. 84, No. HYI, pp. 1531-1 to 1531-27, February 1958.

32. U.S. Bureau of Reclamation, Design of Small Dams, p. 207, Washington, D.C., 1965.

33. U.S. Army Corps of Engineers, "Slope Protection for Earth Dams," Waterways Experiment Station, Vicksburg, Mississippi, 1949.

34. U.S. Army Corps of Engineers, "Hydraulic Design Criteria," Waterways Experiment Station, Vicksburg, Mississippi, 1961 (Revised 1970).

35. U.S. Bureau of Reclamation, "Design of Small Dams," p. 32, p. 50, and pp. 413-431, Washington, D.C., 1965.

36. U.S. Army Corps of Engineers, "Flood-Hydrograph Analyses and Computations," EM 1110-2-1405, August 1959.

37. "Design of Small Dams," Appendix H, p. 787, U.S. Dept. of the Interior, Bureau of Reclamation, Second Edition, 1973.

38. U.S. Army Corps of Engineers, "Flow Through a Breached Dam," Military Hydrograph Bulletin No. 4, 1957.

39. Carl R. Wilder, Director, Department of Energy and Water Resources, Portland Cement Association, Personal Communication, February 1977.

40. W. G. Holtz and F. C. Walker, "Soil-Cement as Slope Protection of Earth Dams," Proceeding, ASCE, Vol. 88, No. SM6, December 1962.

41. Deleted.

42. Deleted.

43. V. T. Chow, "Handbook of Applied Hydrology," Chapter 23, McGraw-Hill Book Co., New York, 1964.

44. U.S. Weather Bureau, "Climatography of the United States," No. 86-9, Washington, D.C., 1964.

45. W. F. Meents, "Glacial-Drift Gas in Illinois," Circular 292, Illinois State Geological Survey, Urbana, Illinois, 1960.

46. W. F. Meents, "Report of Gas Flow Measurement," DeWitt County 26-20N-3E, Illinois State Geological Survey, Urbana, Illinois, September 26, 1974.

47. llinois State Water Survey, open file.

48. D. A. Stephenson, "Hydrogeology of Glacial Deposits of the Mahomet Bedrock Valley in East-Central Illinois," Circular 409, Illinois State Geological Survey, Urbana, Illinois, 1967.

49. C. S. Hunt and J. P. Kempton, "Geology for Planning in DeWitt County," Environmental Geology Notes No. 83, Illinois State Geological Survey, Urbana, Illinois, 1977.

50. W. C. Walton, "Ground-Water Recharge and Runoff in Illinois," Report of Investigation 48, Illinois State Water Survey, Urbana, Illinois, 1965.

51. A. P. Visocky and R. J. Schicht, "Ground-Water Resources of the Buried Mahomet Bedrock Valley," Report of Investigation 62, Illinois State Water Survey, Urbana, Illinois, 1969.

52. E. W. Sanderson, "Groundwater Availability in Piatt County," Circular 107, Illinois State Water Survey, Urbana Illinois, 1971.

53. S. Csallany, "Yields of Wells in Pennsylvanian and Mississippian Rocks in Illinois," Report of Investigation 55, Illinois State Water Survey, Urbana, Illinois, 1966.

54. Illinois Department of Business and Economic Development, "Technical Appendix to Report on Priority Elements for Developing Illinois Water Resources," pp. 14 and 30, HUD Project No. Ill, P-260, 1970.

55. NUREG 1844, Safety Evaluation Report for an Early Site Permit (ESP) at the Exelon Generation Company, LLC (EGC) ESP Site, May 2006.

2.4.15.1 Additional References (Not Cited)

1. O. G. Lara, "Low Flow Frequencies of Illinois Streams," Illinois Division of Waterways and U.S. Geological Survey, 1970.
2. U.S. Bureau of Reclamation, Hydraulic Design of Stilling Basins and Energy Dissipators, Engineering Monograph No. 25, 1964.
3. American Geological Institute, "Glossary of Geology," 1972 Edition, Washington, D.C.
4. P. C. Heigold, "Notes on the Earthquake of November 9, 1968, in Southern Illinois," Env. Geol. Notes 24, Illinois State Geological Survey.
5. S. A. Changnon, Jr., "Climatology of Severe Winter Storms in Illinois," State Water Survey Bulletin 53, Urbana, Illinois, 1969.
6. V. T. Chow, "Handbook of Applied Hydrology," Chapters 10 and 14, McGraw-Hill Book Co., New York, 1964.
7. H. W. King and E. F. Brater, "Handbook of Hydraulics," McGraw-Hill Book Co., 1963.
8. H. J. Tracy, "Discharge Characteristics of Broad-Crested Weirs," U.S.G.S. Circular 397, 1957.
9. U.S. Army Corps of Engineers, "Snow Hydrology," Summary Report of Snow Investigations, June 1956.
10. U.S. Army Corps of Engineers, "Runoff From Snowmelt," January 1960.
11. U.S. Army Corps of Engineers, "Policies and Procedures Pertaining to Determination of Spillway Capacities and Freeboard Allowances for Dams," Engineering Circular (EC) No. 1110-2-27, February 19, 1968.
12. U.S. Army Corps of Engineers, Military Hydrology R&D Branch, "Flow Through a Breached Dam - Military Hydrograph," Bulletin No. 9.
13. U.S. Army Corps of Engineers, "Hydraulic Design of Flood Control Channels," EM 1110-2-1601, July 1970.
14. U.S. Bureau of Reclamation, "Canals and Related Structures," Design Standards No. 3, December 1967.
15. C. V. Davis and K. E. Sorensen, "Handbook of Applied Hydraulics," McGraw-Hill Book Co., New York, 1969.
16. Illinois State Water Survey, Public Groundwater Supplies, open file.
17. U.S. Army Corps of Engineers, “Waves and Wind Tides in Shallow Lakes and Reservoirs,” Summary Report, 1955.
18. R. L. Weigel, “Oceanographical Engineering,” Prentice Hall, New Jersey, 1964.

2.4.15.2 Agencies Contacted

1. Clinton Water Commission (Clinton, Illinois).
2. Illinois Division of Water Resources.
3. Illinois Environmental Protection Agency.
4. Illinois State Water Survey.
5. U.S. Army Corps of Engineers.
6. U.S. Environmental Protection Agency.

TABLE 2.4-1
MEAN MONTHLY RUNOFF, RAINFALL, AND NATURAL LAKE EVAPORATION DATA FOR SALT CREEK BASIN

MONTH	MEAN RUNOFF (inches)	MEAN RAINFALL (inches)	MEAN LAKE EVAPORATION (inches)
January	0.98	2.02	0.48
February	1.05	1.99	0.78
March	1.27	3.01	1.81
April	1.77	3.87	3.12
May	1.50	4.03	4.71
June	1.12	4.98	5.69
July	0.57	3.69	6.35
August	0.24	3.23	5.18
September	0.13	3.24	3.67
October	0.21	3.13	2.33
November	0.34	2.67	1.07
December	0.65	1.84	0.48
TOTAL	9.83	37.70	35.67

TABLE 2.4-2
OBSERVED PEAK DISCHARGES ON SALT CREEK
NEAR ROWELL LARGER THAN 10,000 CFS

DATE	DISCHARGE (cfs)	GAUGE HEIGHTS STAGE (ft)	ELEVATION (ft)
May 18, 1943	12,400	24.77	634.77
May 28, 1956	10,300	23.67	633.67
May 8, 1961	10,300	23.72	633.72
April 21, 1964	10,600	24.71	634.71
May 16, 1968	24,500	29.21	639.21

TABLE 2.4-3
RAINFALL DATA FOR SELECTED MAJOR STORMS IN MIDWEST
AND PROBABLE MAXIMUM STORMS

		MAXIMUM PRECIPITATION (inches)	DURATION (hours)	MAXIMUM AVERAGE DEPTH OF RAINFALL FOR 300-MI^2 (inches) DURATION (hours) 6	12	18	24	36	48
1. Bonaparte, Iowa Le Harpe, Ill.	June 9-10, 1905	12.1	12	8.8	11.1	11.1	11.1	11.1	11.1
2. Bellefontaine, Ohio	Mar. 23-27, 1913	11.2	120	3.3	5.3	6.2	6.8	7.8	8.9
3. Galva, Ill.	Aug. 18-20, 1924	9.2	54	5.9	6.1	7.5	8.2	8.2	8.2
4. Boyden, Iowa	Sept. 17-19, 1926	24.0	54	11.0	14.7	15.5	15.5	15.5	15.5
5. Cole Camp, Mo.	Aug. 12-15, 1946	19.4	78	7.9	8.8	9.0	11.7	15.4	16.8
Probable Maximum Precipitation At Site			48	16.0	19.7	21.2	22.6	24.0	25.2

TABLE 2.4-4
MONTHLY PROBABLE MAXIMUM PRECIPITATION
FOR 24-HOUR DURATION FOR ZONE 7

MONTH	24-HOUR PMP FOR 200 mi^2 (INCHES)	CONVERSION FACTOR FOR AREAS LESS THAN 10 mi^2	24-HOUR PMP FOR AREAS LESS THAN 10 mi^2
All-Season	24.0	1.30	31.20
January	10.5	1.11	11.66
February	10.8	1.14	12.31
March	11.7	1.18	13.81
April	14.6	1.19	17.37
May	18.0	1.26	22.68
June	22.0	1.30	28.60
July	24.0	1.30	31.20
August	24.0	1.30	31.20
September	22.8	1.26	28.73
October	20.0	1.22	24.40
November	14.9	1.14	16.99
December	11.6	1.11	12.88

TABLE 2.4-5
48-HOUR LOCAL PROBABLE MAXIMUM PRECIPITATION
6-HOUR INCREMENTS

6-HOUR TIME PERIOD	CUMULATIVE TIME (HOUR)	INCREMENTAL RAINFALL (INCHES)	CUMULATIVE RAINFALL (INCHES)
1	6	0.09	0.09
2	12	0.33	0.42
3	18	1.88	2.30
4	24	0.10	2.40
5	30	1.20	3.60
6	36	4.32	7.92
7	42	24.48	32.40
8	48	1.20	33.60

TABLE 2.4-6
TIME DISTRIBUTION OF MAXIMUM 6-HOUR RAINFALL

RAINFALL PERIODS OF 30-MINUTES FOR 6-HOUR PERIOD	PERCENT OF TOTAL 6-HOUR RAINFALL
1	3
2	3
3	4
4	5
5	6
6	12
7	43
8	8
9	6
10	4
11	3
12	3

Note: This is for 6-hour interval only with the maximum rainfall of 43% for the seventh 1/2-hour interval of 24.48 inches in 6 hours.

TABLE 2.4-7
PROBABLE MAXIMUM PRECIPITATION
DEPTH - DURATION

	DEPTH OF RAINFALL (inches)			
	DURATION (hours)			
MONTH	6	12	24	48
January	4.3	7.3	10.1	13.0
February	4.6	7.9	10.7	13.8
March	5.7	9.4	11.4	14.4
April	8.7	11.7	14.2	17.1
May	12.5	15.0	17.3	21.0
June	15.4	18.1	21.2	24.0
July	15.8	19.6	22.4	25.0
August	16.0	19.7	22.6	25.2
September	15.5	19.3	21.8	24.2
October	13.1	16.1	19.2	22.0
November	9.3	12.1	14.2	17.4
December	5.7	8.4	10.6	13.8
All Season	16.0	19.7	22.6	25.2

TABLE 2.4-8
PROBABLE MAXIMUM PRECIPITATION FOR VARIOUS DURATIONS

DURATION (hours)	200-mi^2 24-hr PRECIPITATION FOR AUGUST (inches)	PRECENT OF 200-mi^2 24-hr VALUE	TOTAL PRECIPITATION FOR 296-mi^2 (inches)	24-HOUR INCREMENTAL PRECIPITATION (inches)
6		68	15.98	
12		84	19.74	
24	23.5	96	22.56	22.56
48		107	25.15	2.59

TABLE 2.4-9
RAINFALL DISTRIBUTION

6-HOUR PERIOD	PERCENTAGE	1ST 24 HOURS	2ND 24 HOURS
4	5.3	1.20	0.14
2	13.8	3.11	0.36
1	72.5	16.36	1.87
3	8.4	1.89	0.22
TOTAL	100.0	22.56	2.59

TABLE 2.4-10
2-HOURLY PROBABLE MAXIMUM PRECIPITATION (PMP) FOR 48-HOUR PERIOD

TIME (HOURS)	6-HOURLY PRECIPITATION of 24-HOUR PERIOD (FROM TABLE 2.4-9 2ND 24 HOURS)	2-HOURLY PERCENTAGE OF EACH 6-HOUR PRECIPITATION	1 PRECIPITATION (INCHES)		LOSSES (INCHES) INITIAL LOSS = 0 INFILTRATIONS =.1 in/hr	RAINFALL EXCESS (INCHES)	
			INCREMENTAL	CUMULATIVE		INCREMENTAL	CUMULATIVE
0	0	0	0	0	0	0	0
2		0.26	0.04	0.04	0.2	0	0
4		0.53	0.07	0.11	0.2	0	0
6	0.14	0.21	0.03	0.14	0.2	0	0
8		0.26	0.09	0.23	0.2	0	0
10		0.53	0.19	0.42	0.2	0	0
12	0.36	0.21	0.08	0.50	0.2	0	0
14		0.26	0.49	0.94	0.2	0.29	0.29
16		0.53	0.99	1.98	0.2	0.79	1.08
18	1.87	0.21	0.39	2.37	0.2	0.19	1.27
20		0.26	0.06	2.43	0.2	0	0
22		0.53	0.12	2.55	0.2	0	1.27
24	0.22	0.21	0.05	2.60	0.2	0	1.27
26		0.26	0.31	2.91	0.2	0.11	1.27
28		0.53	0.64	3.55	0.2	0.44	1.82
30	1.20	0.21	0.25	3.80	0.2	0.05	1.87
32		0.26	0.81	4.61	0.2	0.61	2.48
34		0.53	1.65	6.26	0.2	1.45	3.93
36	3.11	0.21	0.65	6.91	0.2	0.45	4.38
38		0.26	4.26	11.19	0.2	4.06	8.44
40		0.53	8.67	19.84	0.2	8.47	16.91
42	16.36	0.21	3.43	23.27	0.2	3.23	20.43
44		0.26	0.49	23.27	0.2	0.29	20.14
46		0.53	1.00	24.75	0.2	0.80	21.23
48	1.89	0.21	0.40	25.16	0.2	0.20	21.43

TABLE 2.4-11

2-HOURLY ANTECEDENT STANDARD PROJECT STORM FOR 48-HOUR PERIOD

(Standard Project Storm is 50% of Probable Maximum Precipitation)

TIME (INCHES) (HOURS)	PRECIPITATION (INCHES)		LOSSES (INCHES) INITIAL LOSS = 1.5 INCHES INFILTRATION =	RAINFALL EXCESS	
	INCREMENTAL*	CUMULATIVE	0.1 in/hr	INCREMENTAL	CUMULATIVE
2	0.02	0.02		0	0
4	0.04	0.06		0	0
6	0.02	0.08		0	0
8	0.05	0.13		0	0
10	0.10	0.23		0	0
12	0.04	0.27	1.5 of initial	0	0
14	0.25	0.52	loss is	0	0
16	0.50	1.02	satisfied.	0	0
18	0.20	1.22		0	0
20	0.03	1.25		0	0
22	0.06	1.31		0	0
24	0.02	1.33		0	0
26	0.16	1.49		0	0
28	0.32	1.81	0.2	0.12	0.12
30	0.12	1.93	0.2	0	0.12
32	0.40	2.33	0.2	0.20	0.32
34	0.82	3.15	0.2	0.62	0.94
36	0.33	3.48	0.2	0.13	1.07
38	2.13	5.61	0.2	1.93	3.00
40	4.33	9.94	0.2	4.13	7.13
42	1.72	11.66	0.2	1.52	8.65
44	0.25	11.91	0.2	0.05	8.70
46	0.50	12.41	0.2	0.30	9.00
48	0.20	12.61	0.2	0	9.00

*These values are 50% of PMP in Table 2.4-10.

TABLE 2.4-12
UNITGRAPH CHARACTERISTICS

SPECIFICATIONS	AREA (mi^2)	DURATION OF UNIT HYDROGRAPH (hr)	LAG TIME (hr)	DURATION /LAG RATIO	FORM NO. (Ref. 1)	qs (cfs - internvals)	TIME OF PEAK (hr)	PEAK DISCHARGE (cfs)	TIME BASE tp (hr)
1. Salt Creek Component Areas									
1.1 Headwater Area	126.8	2	19.1	0.1	80.10	42,000	10.5	4,490	78
1.2 Local Areas*									
I_E	6.2	1/2	3.1	0.16	80.15	8,600	2.1	1,275	12
II_E	5.0	1/2	2.8	0.179	80.20	5,760	2.0	1,155	22
III_W	16.3	1	5.6	0.179	80.20	9,400	3.9	1,880	22
IV_W	8.2	1/2	3.7	0.135	80.15	9,500	2.5	1,410	15
	35.5 or 36								
Local Area		2					4.0	4,260	30
2. North Fork Component Areas									
2.1 Headwater Area	111.0	2	17.85	0.11	80.10	40,400	10	4,250	55
2.2 Local Area	15.0	1/2	5.3	.09	80.10	18,000	2.92	1,890	22
Local Area							4	1.718	
3. Reservoir Surface Area	8	2						2,500	2
4. Salt Creek and North Fork Areas Upstream of Dam Site	296	2	32	0.625	500.10	60,000	24	5,690	100

* By applying S-Curve method to 1/2 hour and 1 hour unit hydrographs, 2 hour unit hydrographs were developed for local areas. Summation of the ordinates of these curves gives 2 hour unit hydrographs for local areas. Local area I_E is at "SE" side of lake, II_E is located on "NE" side, III_W is at "NW" side and IV_W is at "SW" side of lake

TABLE 2.4-13
PROBABLE MAXIMUM HYDROGRAPH AT DAM SITE
UNDER NATURAL RIVER CONDITIONS
Drainage Area - 296 mi^2

TIME (hr)	DISCHARGE FROM DRAINAGE AREA (cfs)	BASEFLOW (cfs)	TOTAL DISCHARGE (cfs)
2.	0.	250.	250.
4.	0.	250.	250.
6.	0.	250.	250.
8.	0.	250.	250.
10.	0.	250.	250.
12.	0.	250.	250.
14.	0.	250.	250.
16.	0.	250.	250.
18.	0.	250.	250.
20.	0.	250.	250.
22.	0.	250.	250.
24.	0.	250.	250.
26.	0.	250.	250.
28.	30.	250.	280.
30.	80.	250.	330.
32.	164.	250.	414.
34.	457.	250.	707.
36.	866.	250.	1116.
38.	1724.	250.	1974.
40.	4087.	250.	4337.
42.	7282.	250.	7532.
44.	10569.	250.	10819.
46.	14717.	250.	14967.
48.	19205.	250.	19455.
50.	24349.	250.	24599.
52.	30586.	250.	30896.
54.	36200.	250.	36450.
56.	41253.	250.	41503.
58.	45702.	250.	45952.
60.	48516.	250.	48766.
62.	49729.	250.	49979.
64.	49097.	250.	49347.
66.	47128.	250.	47378.
68.	44829.	250.	45079.
70.	41826.	250.	42076.
72.	38477.	250.	38727.
74.	35181.	250.	35431.
76.	31425.	250.	31675.
78.	28130.	250.	28380.
80.	25351.	250.	25601.
82.	22888.	250.	23138.
84.	20714.	250.	20964.
86.	18478.	250.	18728.
88.	16635.	250.	16885.
90.	14955.	250.	15205.
92.	13263.	250.	13513.
94.	11686.	250.	11936.
96.	9976.	250.	10226.

TABLE 2.4-13 (CONT'D)

TIME (hr)	DISCHARGE FROM DRAINAGE AREA (cfs)	BASEFLOW (cfs)	TOTAL DISCHARGE (cfs)
98.	8653.	250.	8903.
100.	5864.	250.	6114.
102.	1789.	250.	2039.
104.	375.	250.	625.
106.	270.	250.	520.
108.	0.	250.	250.
110.	0.	250.	250.
112.	0.	250.	250.
114.	0.	250.	250.
116.	0.	250.	250.
118.	0.	250.	250.
120.	0.	250.	250.
122.	0.	250.	250.
124.	0.	250.	250.
126.	0.	250.	250.
128.	0.	250.	250.
130.	0.	250.	250.
132.	0.	250.	250.
134.	72.	250.	322.
136.	392.	250.	642.
138.	852.	250.	1102.
140.	1284.	250.	1534.
142.	1837.	250.	2087.
144.	2457.	250.	2707.
146.	3211.	250.	3461.
148.	4829.	250.	4579.
150.	5399.	250.	5649.
152.	6538.	250.	6788.
154.	8146.	250.	8396.
156.	9804.	250.	10054.
158.	12231.	250.	12481.
160.	17526.	250.	17776.
162.	24342.	250.	24592.
164.	31468.	250.	31718.
166.	40330.	250.	40580.
168.	49835.	250.	50085.
170.	60693.	250.	60943.
172.	73516.	250.	73766.
174.	85129.	250.	85379.
176.	95576.	250.	95826.
178.	104690.	250.	104940.
180.	110408.	250.	110658.
182.	112677.	250.	112927.
184.	111071.	250.	111321.
186.	106696.	250.	106946.
188.	101409.	250.	101659.
190.	94655.	250.	94905.
192.	87120.	250.	87370.
194.	79647.	250.	79897.
196.	71023.	250.	71273.
198.	63016.	250.	63266.
200.	56631.	250.	56381.

TABLE 2.4-13 (CONT'D)

TIME	DISCHARGE FROM DRAINAGE	BASEFLOW (cfs)	TOTAL DISCHARGE (cfs)
202.	51130.	250.	51380.
204.	46213.	250.	46463.
206.	41253.	250.	41503.
208.	37029.	250.	37279.
210.	33124.	250.	33374.
212.	29354.	250.	29604.
214.	25718.	250.	25968.
216.	21873.	250.	22123.
218.	18862.	250.	19112.
220.	12934.	250.	13184.
222.	4478.	250.	4728.
224.	1385.	250.	1635.
226.	940.	250.	1190.
228.	180.	250.	430.

TABLE 2.4-14
TOTAL PROBABLE MAXIMUM FLOOD HYDROGRAPH
FOR LAKE CLINTON
Drainage Area - 290 mi^2

TIME (hr)	DISCHARGE FROM DRAINAGE AREA (cfs)	PPT. ON LAKE (cfs)	BASEFLOW (cfs)	TOTAL DISCHARGE (cfs)
2.	0.	52.	250.	302.
4.	0.	103.	250.	353.
6.	0.	52.	250.	302.
8.	0.	129.	250.	379.
10.	0.	258.	250.	508.
12.	0.	103.	250.	353.
14.	0.	645.	250.	895.
16.	0.	1291.	250.	1541.
18.	0.	516.	250.	766.
20.	0.	77.	250.	327.
22.	0.	155.	250.	405.
24.	0.	52.	250.	302.
26.	0.	413.	250.	663.
28.	299.	826.	250.	1375.
30.	891.	310.	250.	1451.
32.	1528.	1136.	250.	2914.
34.	4139.	2117.	250.	6506.
36.	7801.	852.	250.	8903.
38.	13981.	336.	250.	14566.
40.	34254.	852.	250.	35356.
42.	60656.	4440.	250.	65346.
44.	73304.	645.	250.	74199.
46.	78128.	1291.	250.	79669.
48.	79308.	516.	250.	80074.
50.	72380.	0.	250.	72630.
52.	62039.	0.	250.	62289.
54.	53117.	0.	250.	53367.
56.	45349.	0.	250.	45599.
58.	37955.	0.	250.	36205.
60.	31145.	0.	250.	31395.
62.	25509.	0.	250.	25759.
64.	20912.	0.	250.	21162.
66.	17282.	0.	250.	17532.
68.	14621.	0.	250.	14871.
70.	12728.	0.	250.	12978.
72.	11203.	0.	250.	11453.
74.	9818.	0.	250.	10068.
76.	8620.	0.	250.	8870.
78.	7643.	0.	250.	7893.
80.	6762.	0.	250.	7012.
82.	5945.	0.	250.	6195.
84.	5237.	0.	250.	5487.
86.	4642.	0.	250.	4892.
88.	4114.	0.	250.	9364.
90.	3586.	0.	250.	3836.
92.	3226.	0.	250.	3476.
94.	2951.	0.	250.	3201.
96.	2560.	0.	250.	2810.

TABLE 2.4-14 (CONT'D)

TIME	DISCHARGE FROM DRAINAGE	PPT. ON LAKE (cfs)	BASEFLOW (cfs)	TOTAL DISCHARGE (cfs)
98.	2163.	0.	250.	2413.
100.	1788.	0.	250.	2038.
102.	1463.	0.	250.	1713.
104.	1209.	0.	250.	1459.
106.	905.	0.	250.	1155.
108.	630.	0.	250.	880.
110.	371.	0.	250.	621.
112.	113.	0.	250.	363.
114.	25.	0.	250.	275.
116.	15.	0.	250.	265.
118.	0.	0.	250.	250.
120.	0.	0.	250.	250.
122.	0.	103.	250.	353.
124.	0.	181.	250.	431.
126.	0.	77.	250.	327.
128.	0.	232.	250.	482.
130.	0.	490.	250.	740.
132.	0.	207.	250.	457.
134.	724.	1265.	250.	2238.
136.	4125.	2556.	250.	6931.
138.	8830.	1007.	250.	10087.
140.	10863.	155.	250.	11267.
142.	11708.	310.	250.	12268.
144.	11864.	129.	250.	12243.
146.	10996.	800.	250.	12046.
148.	10960.	1652.	250.	12862.
150.	12042.	645.	250.	12937.
152.	13262.	2091.	250.	15603.
154.	19220.	4259.	250.	23729.
156.	27270.	1678.	250.	29198.
158.	40274.	10996.	250.	51520.
160.	81616.	22380.	250.	104246.
162.	135265.	8854.	250.	144369.
164.	161193.	1265.	250.	162708.
166.	171445.	2581.	250.	174276.
168.	174332.	1033.	250.	175615.
170.	160410.	0.	250.	160660.
172.	138569.	0.	250.	138819.
174.	119322.	0.	250.	119572.
176.	102193.	0.	250.	102443.
178.	85686.	0.	250.	85936.
180.	70509.	0.	250.	70759.
182.	57969.	0.	250.	58219.
184.	47691.	0.	250.	47941.
186.	39499.	0.	250.	39749.
188.	33409.	0.	250.	33659.
190.	29027.	0.	250.	29277.
192.	25463.	0.	250.	25713.
194.	22274.	0.	250.	22524.
196.	19533.	0.	250.	19783.
198.	17291.	0.	250.	17541.
200.	15291.	0.	250.	15541.

TABLE 2.4-14 (CONT’D)

TABLE 2.4-14 (CONT'D)

TIME	DISCHARGE FROM DRAINAGE	PPT. ON LAKE (cfs)	BASEFLOW (cfs)	TOTAL DISCHARGE (cfs)
202.	13427.	0.	250.	13677.
204.	11802.	0.	250.	12052.
206.	10429.	0.	250.	10679.
208.	9199.	0.	250.	9449.
210.	8016.	0.	250.	8266.
212.	7194.	0.	250.	7444.
214.	6544.	0.	250.	6794.
216.	5677.	0.	250.	5927.
218.	4797.	0.	250.	5047.
220.	3971.	0.	250.	4221.
222.	3256.	0.	250.	3506.
224.	2677.	0.	250.	2927.
226.	2009.	0.	250.	2259.
228.	1411.	0.	250.	1661.
230.	851.	0.	250.	1101.
232.	301.	0.	250.	551.
234.	96.	0.	250.	346.
236.	55.	0.	250.	305.
238.	10.	0.	250.	260.

TABLE 2.4-15
100-YEAR RAINFALL FOR VARIOUS DURATIONS

DURATION (hours)	ACCUMULATIVE POINT RAINFALL (inches)	AREAL CORRECTION FACTOR	ACCUMULATIVE RAINFALL FOR 296-MI^2 (inches)
6	4.65	0.84	3.91
12	5.50	0.88	4.84
24	6.30	0.915	5.76

TABLE 2.4-16
HOURLY RAINFALL FOR 6-HOUR PERIOD (100-YEAR)

TIME (hours)	PERCENT OF 6-HOUR RAINFALL	ACCUMULATIVE RAINFALL (inches)	INCREMENTAL RAINFALL (inches)	REARRANGED SEQUENCE OF INCREMENTAL RAINFALL
1	49	1.92	1.92	0.31 (6)
2	65	2.50	0.58	0.35 (4)
3	75	2.93	0.43	0.43 (3)
4	84	3.28	0.35	1.92 (1)
5	93	3.60	0.32	0.58 (2)
6	100	3.91	0.31	0.32 (5)

TABLE 2.4-17
100-YEAR STORM DISTRIBUTION

TIME (hours)	INCREMENTAL RAINFALL (inches)	ACCUMULATIVE RAINFALL (inches)
0 - 2	0.66	0.66
2 - 4	2.35	3.01
4 - 6	0.90	3.91
6 - 12	0.93	4.84
12 - 24	0.92	5.76

TABLE 2.4-18

LAND USAGE AND HYDROLOGIC SOIL-COVER COMPLEX NUMBER*

LAND USE OR COVER	PERCENT OF AREA	CURVE NUMBER	PERCENT TIMES NUMBER
Row Crops, Good	50	78	0.50 x 78 = 39.0
Pasture and Forest, Fair	40	69	0.40 x 69 = 27.6
Farmstead and Roads	10	74	0.10 x 74 = 7.4

* Hydrologic soil-cover complex number = 74.0.

TABLE 2.4-19
RAINFALL EXCESS AND INFILTRATION LOSSES
(100-YEAR)

TIME LOSSES (hours)	PRECIPITATION (inches)		RAINFALL EXCESS (inches)		INFILTRATION
	INCREMENTAL	ACCUMULATIVE	ACCUMULATIVE	INCREMENTAL	($in.^2$/HR)
0 - 2	0.66	0.66	0.12	0.12	0.54
2 - 4	2.35	3.01	2.00	1.88	0.47
4 - 6	0.90	3.91	2.82	0.82	0.08
6 - 8	0.49	4.40	3.30	0.44	0.05
8 - 10	0.30	4.70	3.60	0.25	0.05
10 - 12	0.15	4.85	3.75	0.10	0.05
12 - 14	0.20	5.05	3.95	0.15	0.05
14 - 16	0.15	5.20	4.08	0.12	0.03
16 - 18	0.15	5.35	4.22	0.10	0.05
18 - 20	0.15	5.50	4.37	0.10	0.05

TABLE 2.4-20
LOW FLOWS FOR 100-YEAR DROUGHT

DURATION (months)	RUNOFF (inches) CUMULATIVE	INCREMENTAL	DURATION (months)	RUNOFF (inches) CUMULATIVE	INCREMENTAL
1	0.02	0.02	31	5.05	0.15
2	0.04	0.02	32	5.20	0.15
3	0.06	0.02	33	5.35	0.15
4	0.08	0.02	34	5.50	0.15
5	0.10	0.02	35	5.65	0.15
6	0.12	0.02	36	5.80	0.15
7	0.18	0.06	37	6.00	0.20
8	0.25	0.07	38	6.20	0.20
9	0.31	0.06	39	6.40	0.20
10	0.37	0.06	40	6.60	0.20
11	0.44	0.07	41	6.80	0.20
12	0.50	0.06	42	7.00	0.20
13	0.62	0.12	43	7.67	0.67
14	0.73	0.11	44	8.33	0.66
15	0.85	0.12	45	9.00	0.67
16	0.97	0.12	46	9.67	0.67
17	1.08	0.11	47	10.33	0.66
18	1.20	0.12	48	11.00	0.67
19	1.58	0.38	49	11.50	0.50
20	1.97	0.39	50	12.00	0.50
21	2.35	0.38	51	12.50	0.50
22	2.73	0.38	52	13.00	0.50
23	3.12	0.39	53	13.50	0.50
24	3.50	0.38	54	14.00	0.50
25	3.73	0.23	55	15.17	1.17
26	3.97	0.24	56	16.33	1.16
27	4.20	0.23	57	17.50	1.17
28	4.43	0.23	58	18.67	1.17
29	4.67	0.24	59	19.83	1.16
30	4.90	0.23	60	21.00	1.17

TABLE 2.4-21
NET LAKE EVAPORATION DATA*

DURATION (months)	NET LAKE EVAPORATION INCREMENT (inches)		DURATION (months)	NET LAKE EVAPORATION INCREMENT (inches)	
	50-yr R.I.	**100-yr R.I.		50-yr R.I.	**100-yr R.I.
1	6.97	7.00	31	1.00	0.50
2	5.23	5.50	32	-1.00	0.50
3	5.00	7.00	33	-1.50	-5.00
4	4.00	5.50	34	-1.50	-5.00
5	0.90	0.50	35	0.50	4.00
6	-0.70	-3.00	36	0.50	4.00
7	1.60	3.50	37	3.00	3.00
8	1.00	1.50	38	3.00	3.00
9	-1.00	0.00	39	2.00	6.00
10	0.00	0.00	40	2.00	6.00
11	0.00	0.00	41	-0.50	2.50
12	1.00	0.00	42	-0.50	2.50
13	3.50	4.25	43	0.00	2.50
14	3.50	4.25	44	0.00	2.50
15	2.00	6.00	45	-0.50	-2.00
16	2.00	6.00	46	-0.50	-2.00
17	2.00	2.00	47	1.50	7.50
18	2.00	2.00	48	1.50	7.50
19	-0.50	1.50	49	2.00	-2.50
20	-0.50	1.50	50	2.00	-2.50
21	-1.50	-2.50	51	0.50	-2.50
22	-1.50	-2.50	52	0.50	-2.50
23	1.50	3.00	53	2.00	2.50
24	1.50	3.00	54	2.00	2.50
25	2.50	2.00	55	-1.50	-1.25
26	2.50	2.00	56	-1.50	-1.25
27	2.50	4.00	57	-1.00	5.00
28	2.50	4.00	58	-1.00	5.00
29	0.00	2.50	59	-0.50	-6.25
30	0.00	2.50	60	-0.50	-6.25

* Derived from “Lake Evaporation in Illinois” by W. J. Roberts and J. B. Stall.

** The 50-Yr R.I. Evaporation Values were considered applicable to Historic Drought of 1952-1957.

TABLE 2.4-22
AVERAGE MONTHLY FORCED LAKE EVAPORATION DATA

MONTH	FORCED EVAPORATION AT 70% LOAD FACTOR (1400 MW) (acre-feet)
January	790
February	955
March	1070
April	1370
May	1500
June	1600
July	1680
August	1540
September	1500
October	1300
November	1050
December	865

TABLE 2.4-23

LOW FLOWS FOR HISTORIC DROUGHT - 1952-1957 DROUGHT

SOURCE: USGS WATER SUPPLY PAPER, PART 5 (ILLINOIS RIVER BASIN)

SALT CREEK AT ROWELL

YEAR/ MONTH	RUNOFF (inches)	YEAR/ MONTH	RUNOFF (inches)	YEAR/ MONTH	RUNOFF (inches)
1951		1954		1956	
Oct	0.20	Jan	0.13	Jan	0.05
Nov	1.01	Feb	0.08	Feb	0.49
Dec	0.40	Mar	0.14	Mar	0.28
		Apr	0.43	Apr	0.19
1952		May	0.12	May	2.33
Jan	1.29	Jun	0.35	Jun	0.67
Feb	0.87	Jul	0.03	Jul	0.18
Mar	1.90	Aug	0.07	Aug	0.33
Apr	2.47	Sep	0.006	Sep	0.03
May	1.25	Oct	0.05	Oct	0.01
Jun	2.47	Nov	0.02	Nov	0.03
Jul	0.43	Dec	0.03	Dec	0.04
Aug	0.09				
Sep	0.04	1955		1957	
Oct	0.03	Jan	0.18	Jan	0.20
Nov	0.05	Feb	0.53	Feb	0.39
Dec	0.06	Mar	0.57	Mar	0.20
		Apr	0.54	Apr	4.27
1953		May	0.43	May	3.55
Jan	0.11	Jun	0.89	Jun	1.77
Feb	0.20	Jul	0.19	Jul	0.71
Mar	1.60	Aug	0.03	Aug	0.09
Apr	1.57	Sep	0.03	Sep	0.05
May	0.43	Oct	0.20		
Jun	0.52	Nov	0.10		
Jul	0.90	Dec	0.08		
Aug	0.12				
Sep	0.03				
Oct	0.02				
Nov	0.03				
Dec	0.04				

TABLE 2.4-24

LOW FLOWS FOR 50-YEAR RECURRENCE DROUGHT

DURATION	RUNOFF (inches)		DURATION	RUNOFF (inches)	
(months)	CUMULATIVE	INCREMENTAL	(months)	CUMULATIVE	INCREMENTAL
1	0.03	0.03	31	5.75	0.45
2	0.06	0.03	32	6.20	0.45
3	0.09	0.03	33	6.65	0.45
4	0.11	0.02	34	7.10	0.45
5	0.14	0.03	35	7.55	0.45
6	0.17	0.03	36	8.00	0.45
7	0.28	0.11	37	8.33	0.33
8	0.40	0.12	38	8.67	0.34
9	0.51	0.11	39	9.00	0.33
10	0.62	0.11	40	9.33	0.33
11	0.74	0.12	41	9.67	0.34
12	0.85	0.11	42	10.00	0.33
13	1.01	0.16	43	10.83	0.83
14	1.17	0.16	44	11.67	0.84
15	1.33	0.16	45	12.50	0.83
16	1.48	0.15	46	13.33	0.83
17	1.64	0.16	47	14.17	0.84
18	1.80	0.16	48	15.00	0.83
19	2.32	0.52	49	15.33	0.33
20	2.83	0.51	50	15.67	0.34
21	3.35	0.52	51	16.00	0.33
22	3.87	0.52	52	16.33	0.33
23	4.38	0.51	53	16.67	0.34
24	4.90	0.52	54	17.00	0.33
25	4.97	0.07	55	18.17	1.17
26	5.04	0.07	56	19.33	1.16
27	5.10	0.06	57	20.50	1.17
28	5.17	0.07	58	21.67	1.17
29	5.24	0.07	59	22.83	1.16
30	5.30	0.06	60	24.00	1.17

TABLE 2.4-25
COMPARISON OF HISTORIC AND 50-YEAR DROUGHT RUNOFF VALUES

DURATION (months)	RUNOFF (inches) OBSERVED* (HISTORIC)	RUNOFF (inches) DERIVED** (50-YEAR)
6	0.26 (July 1954 to December 1954)	0.17
12	1.46 (January 1954 to December 1954)	0.85
18	2.60 (July 1953 to December 1954)	1.80
24	5.03 (August 1953 to July 1955)	4.90
30	5.47 (August 1953 to December 1955)	5.30
36	8.33 (May 1953 to April 1956)	8.00
42	10.30 (July 1953 to January 1957)	10.00
48	15.43 (January 1953 to December 1956)	15.00

* Reference 4

** Reference 30

TABLE 2.4-26
ESTIMATED STATION WATER REQUIREMENTS

		UNIT 1, NORMAL OPERATION 1007 C.F. SUMMER (gpm)	UNIT 1, NORMAL OPERATION 1007 C.F. WINTER (gpm)	UNIT 1, SHUTDOWN (gpm)	UNIT 1 LOCA (1) (gpm)
I.	Circulating Water System	565,800	445,000(2	565,800(5)	
II.	Plant Service Water System				
	Includes the following uses:	37,500	7,500(4)	37,500	
	A. Turbine Oil Coolers B. TBCCW Heat Exchangers C. Generator Stator Coolers D. Hydrogen Coolers E. CCW Heat Exchangers				
III.	Shutdown Service Water System			10,500	Division 1 ~ 12,300 (6)
	Includes the following uses:				Division 2 ~ 12,300 (6) Division 3 ~ 1,000
	A. Diesel Generators B. ECCS Equip. Area Coolers C. Essential Switchgear Heat Removal D. Control Room HVAC E. SSW Pump Room Coolers F. Fuel Pool G. RHR Heat Exchangers H. Drywell Chillers				

(1) Assumptions: LEOP; SSE.
(2) Two pumps operating at runout conditions.
(3) Deleted.
(4) Assumes 35ºF cooling water temperature.
(5) Will decrease to zero after about 4 hours.
(6) Flow (~2000 GPM) to drywell chillers item III.H above excluded for LOCA.

TABLE 2.4-27
PHYSICAL CHARACTERISTICS OF CPS TEST WELL

Location, state coordinates	N 1,272,000/E 360,150
Surface elevation	Approx. 715 ft MSL
Total depth	358 ft
Diameter	12 inches
Screened intervals	255 to 265 ft and 280 to 340 ft 12 in.
Data completed	August 14, 1974
Pumping Test Data	
Data test began	September 26, 1974
Static water level	113 ft
Pumping water level	143 ft
Pumping rate (final)	1030 gpm
Length of test	26 hr
Specific capacity	34.3 gpm/ft

TABLE 2.4-28
PARTIAL WATER QUALITY ANALYSES FOR CPS TEST WELL

PARAMETER*	A & H ENGINEERING CORPORATION+	ILLINOIS STATE WATER SURVEY**
pH	7.4	not reported
Hardness	279 (EDTA)	264 (as CaC03)
Alkalinity	472 (total)	480 (as CaC03)
Chloride	140	160
Fluoride	not reported	0.7
Sulfate	< 1	not reported
Nitrate	0.13 (as N)	0.9
Silica (as Si02)	19	not reported
Iron (total)	4.5	1.6
Manganese	0.15 (total)	0.08
Total Dissolved Solids	641	784

NOTES:

A gas flow measurement was made during test well pumping on September 26, 1974 by the Illinois State Water Survey. Results of two gas analyses indicated that methane comprised more than 80% of the total gas sample.

* All parameters except pH are reported in milligrams per liter.

+ Water samples were collected on September 26 and 27, 1974, during test pumping of the test well.

** Water sample was collected from the test well on September 11, 1974.

TABLE 2.4-29
STRATIGRAPHIC UNITS AND THEIR HYDROGEOLOGIC CHARACTERISTICS

GEOLOGIC SYSTEM	STRATIGRAPHIC UNIT	DESCRIPTION	HYDROGEOLOGIC SYSTEM	HYDROGEOLOGIC CHACTERISTICS
	Henry Formation	Clayey silt overlying stratified silt, sand or gravel	Alluvium	Groundwater occurs in permeable sand and gravel deposits underlying the fine-grained floodplain deposits. Yields are generally suitable for domestic or farm use. Sufficient quantities for municipal use may be available in those areas along the larger streams where thick sand and gravel deposits are present
	Richland Loess	Clayey silt, trace fine sand	Wisconsinan deposits	Groundwater may be obtained from sand and gravel lenses in the Wisconsinan and Illinoian tills and from Kansan outwash deposits (Banner Formation) in the buried Mahomet Bedrock Valley. Groundwater occurs under water table conditions in the Wisconsinan deposits and under artesian conditions in the Illinoian and Kansan deposits.
	Wedron Formation	Clayey sandy silt till with interbedded dis-continuous lenses of stratified silt, sand, or gravel		
	Robien Silt	Silt, some organics, trace clay and fine sand	Illinoian deposits	Yields from wells that intercept good water yielding sand and gravel deposits are suitable for domestic and farm purposes. Higher yields for small industrial or municipal supply are locally available. Where sand and gravel deposits are thin or absent, small amounts of groundwater may be obtained using large diameter wells
	Glasford Formation	Sandy silt till, with interbedded discontin-uous lenses of strati-fied silt, sand, or-sandy silt; upper 10 ft. is highly weath-ered (altered)		
	Banner Formation	Complex sequence of stratified silt, sand clay till, and sand and gravel outwash	Kansan deposits	Kansan sand and gravel deposits in the buried Mahomet Bedrock Valley comprise the major aquifer in the area. Yields of up to 2000 gpm may be obtained from a suitably constructed well located in the main channel of the valley.
	Bond Formation	Shale with thin beds of limestone, sand-stone, siltstone underclay, and coal	Pennsylvanian bedrock	Groundwater occurs in thin sandstone and fractures limestone beds under artesian con-ditions. Small quantities of groundwater, suitable only for domestic or farm supply, may be obtained from the upper 50 to 100 ft. of the Pennsylvanian formations. Deeper drilling is not recommended because ground-water in the deeper formations is highly mineralized.
	Modesto Formation			
	Carbondale Formation			
	Spoon Formation			
	Abbott Formation			

NOTES:

1. The stratigraphic units are discussed in detail in Subsection 2.5.1.2.

2. Figure 2.5-274 shows a comparison of stratigraphic nomenclature used in the USAR, PSAR and boring logs.

3. Excavations for the Clinton Power Station did not extend below the Glasford Formation.

4. Borings for the Clinton Power Station did not fully penetrate rocks of the Carbondale Formation.

5. The vertical scale does not represent the relative thickness of the stratigraphic units.

TABLE 2.4-30
QUALITY OF GROUNDWATER IN ILLINOIAN AND KANSAN AQUIFERS

	ILLINOIAN AQUIFER		KANSAN AQUIFER	
PARAMETER	(range)	(mean)	(range)	(mean)
Iron	0.6 - 3.2	1.8	0.2 - 3.0	1.2
Chloride	Trace - 37	4.1	Trace - 66	11
Sulfate	0 - 9	2.3	0 - 5	0.9
Alkalinity (as $CaCO_3$)	278 - 475	363	284 - 454	363
Hardness (as $CaC0_3$)	170 - 595	299	150 - 438	293
Total dissolved minerals	310 - 478	379	295 - 636	414

NOTES:

1. All parameters are reported in parts per million
2. Data are from Reference 51.

TABLE 2.4-31
PIEZOMETER INSTALLATION DATA

PIEZOMETER NUMBER	DATE OF INSTALLATION	SURFACE ELEVATION (ft, MSL)	TESTED INTERVAL DEPTH (ft)	TESTED INTERVAL ELEVATION (ft, MSL)	STRATIGRAPHIC UNITS OPEN TO PIEZOMETER
P-1A*	6-26-72	675.9	66.0? to 79.5	596.4 to 609.9?	Illinoian
P-1B*	6-26-72	675.9	10.0? to ?	? to 665.9?	Alluvium
P-7B*	7-5-72	737.5	70.0 to 78.0	659.5 to 667.5	Illinoian
P-17*	7-10-72	738.3	149.0 to 240.0	498.3 to 589.3	Illinoian and Kansan
P-20*	6-28-72	738.3	170.0 to 305.5	432.8 to 568.3	Illinoian Kansan, and Bedrock
P-22B*	6-28-72	734.0	55.0 to 64.0	670.0 to 679.0	Illinoian
P-27*	6-6-72	742.9	57.5	85.4	Illinoian
P-31*	9-11-73	736.8	50.0 to 159.0	577.8 to 686.8	Illinoian
P-36*	11-6-73	738.2	178.0 to 223.0	515.2 to 560.2	Kansan
P-37*	8-27-73	739.1	16.0 to 40.0	699.1 to 723.1	Wisconsinan
P-39*	8-28-73	740.8	62.0 to 150.0	590.8 to 678.8	Illinoian
P-40*	10-19-73	742.1	10.0 to 38.0	704.1 to 732.1	Wisconsinan
D-3A*	7-13-72	660.0	30.0 to 40.0	620.0 to 630.0	Illinoian
D-3B*	7-13-72	660.0	10.5 to 20.5	639.5 to 649.5	Alluvium
D-8B*	7-19-72	655.7	1.5 to 16.0	639.7 to 654.2	Alluvium
D-11*	6-21-72	653.8	140.0 to 343.5	310.3 to 513.8	Kansan, Mahomet Sand, and Bedrock
D-19A*	7-13-72	658.9	33.0 to 38.0	620.9 to 625.9	Illinoian
D-19B*	7-13-72	658.9	23.0 to 30.0	628.9 to 635.9	Alluvium and Illinoian
D-23A*	7-14-72	655.8	25.0 to 31.5	624.3 to 630.8	Illinoian
D-23B*	7-14-72	655.8	11.5 to 16.0	639.8 to 644.3	Alluvium
D-30B*	7-26-72	669.9	3.5 to 12.0	657.9 to 666.4	Alluvium
D-30C*	7-27-72	669.9	45.0 to 50.0	619.9 to 624.9	Illinoian
D-31*	6-16-72	667.7	158.0 to 356.5	311.2 to 509.7	Illinoian,

TABLE 2.4-31 (CONT'D)

PIEZOMETER NUMBER	DATE OF INSTALLATION	SURFACE ELEVATION (ft, MSL)	TESTED INTERVAL DEPTH (ft)	TESTED INTERVAL ELEVATION (ft, MSL)	STRATIGRAPHIC UNITS OPEN TO PIEZOMETER
					Mahomet Sand, and Bedrock
D-46	4-24-73	710.3	2.0 to 27.0	683.3 to 708.3	Wisconsinan and Illinoian
D-47*	4-24-73	714.8	2.0 to 38.0	676.8 to 712.8	Wisconsinan and Illinoian
D-48	4-24-73	715.3	2.0 to 39.0	676.3 to 713.3	Wisconsinan and Illinoian
D-50	4-30-73	718.0	2.0 to 37.0	681.0 to 716.0	Wisconsinan
E-1B*	7-13-72	733	30 to 40	693 to 703	Wisconsinan
E-2B*	7-12-72	746	60.68	678 to 686	Illinoian
E-3A	7-5-72	730	214 to 238	492 to 516	Kansan and Mahomet Sand
E-3B	7-12-72	730	68 to 75	655 to 662	Illinoian
E-4B	7-6-72	740	80 to 96	644 to 654	Illinoian
E-5B	7-19-72	750	70 to 76	674 to 680	Illinoian
E-6B	7-25-72	736	0 to 151	585 to 736	Wisconsinan, Illinoian, and Kansan
E-7*	7-20-72	712	0 to 151	560.5 to 712	Wisconsinan, Illinoian, and Kansan
OW-1	5-12-76	716.7	60 to 70	646.7 to 656.7	Illinoian
OW-2*	5-12-76	-	5 to 20	-	Wisconsinan and Illinoian
OW-3s	5-10-76	735.9	5 to 10	725.9 to 730.9	Wisconsinan
OW-3d	5-10-76	735.9	10 to 40	695.9 to 725.9	Wisconsinan
OW-4s	5-7-76	720.9	2.5 to 6.5	714.1 to 718.1	Wisconsinan
OW-4d	5-7-76	721.0	10 to 23.5	697.5 to 711.0	Wisconsinan
OW-5s	5-7-76	712.8	4 to 8	704.8 to 708.8	Wisconsinan
OW-5d	5-7-76	712.6	10 to 18.2	694.4 to 702.6	Wisconsinan

TABLE 2.4-31 (CONT'D)

PIEZOMETER NUMBER	DATE OF INSTALLATION	SURFACE ELEVATION (ft, MSL)	TESTED INTERVAL DEPTH (ft)	TESTED INTERVAL ELEVATION (ft, MSL)	STRATIGRAPHIC UNITS OPEN TO PIEZOMETER
OW-6s	5-10-76	743.3	2.5 to 7.5	735.8 to 740.8	Wisconsinan
OW-6d	5-10-76	743.2	10 to 52	691.2 to 733.2	Wisconsinan
OW-7s	5-13-76	718.6	2 to 6	712.6 to 716.6	Wisconsinan
OW-7d	5-13-76	718.6	10 to 25	693.6 to 708.6	Wisconsinan
OW-8	5-12-76	719.2	18 to 42	677.2 to 701.2	Wisconsinan and Illinoian
OW-9	8-1-77	654.3	16.5 to 24.5	629.8 to 637.8	Illinoian
OW-10	8-2-77	656.0	27.0 to 35.0	621.0 to 629.0	Illinoian
OW-11	8-2-77	654.5	19.0 to 27.0	627.5 to 635.5	Illinoian
OW-12	8-2-77	659.2	17.0 to 25.0	634.2 to 642.2	Alluvium and Illinoian
OW-13	8-2-77	662.1	32.0 to 40.0	622.1 to 630.1	Illinoian
OW-14	8-2-77	657.1	23.0 to 31.0	626.1 to 634.1	Illinoian
OW-15	8-3-77	664.5	47.0 to 55.0	609.5 to 617.5	Illinoian
OW-16	8-3-77	657.9	22.0 to 30.0	627.9 to 635.9	Illinoian
OW-17	8-3-77	659.5	32.0 to 40.0	619.5 to 627.5	Illinoian
OW-18	7-16-79	656.5	7.0 to 15.0	641.5 to 649.5	Alluvium and Fill
OW-19	7-16-79	654.5	6.0 to 18.5	636.0 to 648.5	Alluvium and Illinoian
OW-20	7-17-79	658.4	10.0 to 34.4	624.0 to 648.4	Illinoian and Fill
OW-21	10-8-79	670.0	5.0 to 55.0	615.0 to 665.0	Illinoian and Fill
OW-22A	10-9-79	665.9	23.0 to 44.5	621.4 to 642.9	Illinoian
OW-22B	10-9-79	665.9	5.5 to 20.0	645.9 to 660.4	Wisconsinan
OW-23	10-10-79	654.5	5.0 to 34.5	620.0 to 649.5	Illinoian and Fill
OW-24	10-11-79	654.9	5.0 to 34.0	620.9 to 649.9	Illinoian and Fill

TABLE 2.4-31 (CONT'D)

NOTES:

1. Locations of existing piezometers are shown in Figures 2.4-32 and 2.5-272.

2. Measured ground water level for all piezometers except P-31, P-36, P-37, P-39, and P-40 are shown in Figures 2.4-36 through 2.4-43 and 2.4-48 through 2.4-50. Water level elevations are listed in Table 2.4-36 for those piezometers not included on the figures.

3. "Tested Interval" refers to portion of piezometer backfilled with pea gravel and open to stratigraphic unit.

* Piezometer was destroyed by construction activities.

TABLE 2.4-32
CHEMICAL ANALYSES OF GROUNDWATER SAMPLES FROM SELECTED PIEZOMETERS

PIEZOMETER NUMBER	TO WATER (feet)	TESTED AQUIFER	pH	Ca++	Mg++	Na+	K+	C1-	SO4--	CO3--	HCO_3-	Fe	SiO_2
D-3A	10	Illinoian	6.77	40	49	60	9	37	180	0	122	*	19
D-31	60	Illinoian, Kansan, and Bedrock	9.98	35	2	18	4	40	60	36	0	*	30
E-3A	10	Kansan	7.25	36	41	46	5	15	65	0	195	*	23
E-5B	19	Illinoian	7.01	43	38	100	8	35	30	0	262	0.32	28
E-6	12	Wisconsinan, Illinoian, and Kansan	6.62	62	82	24	2	12	325	0	278	*	26
E-7	5	Wisconsinan and Illinoian	6.70	17	28	16	3	12	55	0	55	*	5
P-14	60	Illinoian and Kansan	6.90	39	45	11	1	15	30	0	149	0.10	19
P-22	30	Illinoian	7.19	46	42	34	4	14	70	0	214	*	18

NOTES:

1. All concentrations except pH are reported in parts per million (ppm).

2. Locations of the P-series piezometers are shown in Figure 2.5-16. Remaining piezometers are shown in Figures 2.4-32.

* Concentration was below the detection limit of about 0.1 ppm.

TABLE 2.4-33
NON-PRIVATE WATER WELLS WITHIN 15 MILES OF SITE

WELL NUMBER	OWNER		DEPTH (ft)	DATE DRILLED	PUMP TEST DATA DATE	STATIC WATER LEVEL (ft)	PUMPING RATE (gpm)	DRAW-DOWN (FT)	CASING (ft)	PERFORATIONS (ft)	AQUIFER
129	City of	(#1)	327	1913	1938	150	250	20	-	-	-
	Clinton	(#2)	358	1914	1941	-	267	-	-	-	-
		(#3)	360	1923	-	-	-	-	340	340-360	-
		(#4)	345	1948	1948	101	204	25	308	308-328	-
		(#5)	361	1946	1946	119	410	19	321	321-361	Sand-Gravel
130	Empire School		68	-	1945	12	6	5	65	65-68	Sand-Gravel
132	Village of	(#1)	165	1923	1924	40	-	-	153	153-165	Sand
	Weldon	(#2)	164	-	1948	67	60	58	154	154-164	Sand
133	Village of Weldon	(#3)	167	1963	1963	76	100	32	157	157-167	Sand
134	Village of	(#4)									
	Weldon		163	1972	1972	91	15	65	157	157-163	Silty Sand
135	Village of	(#5)	170	1972	1972	91	10	3.2	157	157-163	Sand-Gravel
	Weldon	(Test)	166	1971	1971	88	36	17	153	153-166	Sand
136	DeWitt County		22	-	1934	15	5	5	-	-	Gravel
137	DeWitt County		193	-	1965	70	12	-	-	-	Sand-Gravel
138	Weldon Springs	(#1)	55	1949	-	-	-	-	-	-	-
	State Park	(#3)	67	1957	-	-	-	-	-	-	-
		(#4)	73	1950	-	-	-	-	-	-	-
		(#5)	60	1959	1959	36	12	6	57	57-60	Sand-Gravel
		(#6)	75	1970	-	-	-	-	-	-	-
		(#8)	38	1955	1955	1	15	5	-	-	Sand-Gravel
		(#9)	140	1975	1975	39	15	5	132	132-140	Sand
139	High School (Heyworth)		77	-	-	-	10	-	-	-	-
140	Farmer City	(#1)	173	1932	-	-	-	-	156	156-173	Sand-Clay
		(#2)	167	1945	1945	59	67	46	152	152-167	Sand-Gravel
141	Wapella City	(#1)	78	1941	1975	15	100	10.3	74	74-78	Sand-Gravel
		(#2)	79	1950	1950	18	200	-	69	69-79	-
142	Heyworth	(Test)	106	1935	-	-	-	-	-	-	-
143	Heyworth	(Test)	300	1935	-	-	-	-	-	-	-
144	Wapella High School		311	1933	-	-	-	-	-	-	-
145	City of	(Test)	360	1954	-	-	-	-	-	-	-

TABLE 2.4-33 (CONT'D)

WELL NUMBER	OWNER		DEPTH (ft)	DATE DRILLED	PUMP TEST DATA DATE	STATIC WATER LEVEL (ft)	PUMPING RATE (gpm)	DRAW-DOWN (FT)	CASING (ft)	PERFORATIONS (ft)	AQUIFER
	Clinton										
146	City of	(Test)	349	1954	-	-	-	-	-	-	-
	Clinton										
147	City of	(Test)	343	1954	-	-	-	-	-	-	-
	Clinton										
148	City of	(Test)	372	1954	-	-	-	-	-	-	-
	Clinton										
149	City of	(#6)	345	1948	1960	106	500	22	305	305-345	-
	Clinton	(#7)	345	1954	1955	111	940	20	305	305-345	Sand-Gravel
		(#8)	352	1973	1973	105	620	10	276	276-338	Sand-Gravel
150	Bosserman School		73	1939	1939	8	4	32	-	-	-
151	Progress School		49	1938	1938	20	8	0	46	46-49	Gravel
152	City of Clinton		320	1971	-	-	-	-	-	-	-
153	Clinton Theater		130	1936	1936	3	50	100	-	124-130	Sand-Gravel
154	City of Clinton	(Test)	349	1971	-	-	-	-	-	-	-
170	DeWitt		340	1974	1974	112	400	60	300	300-340	Sand-Gravel
171	Farmer City	(#3)	172	1951	1952	68	135	26	158	158-172	
		(#4)	167	1954	1974	54	10	16	152	152-167	Sand-Gravel
		(#6)	172	1955	1975	61	200	43	152	152-172	Sand-Gravel
172	Farmer City	(#5)	150	1955	1955	40	150	85	125	125-150	Sand-Gravel
		(#8)	152.5	1972	1974	84	36	24	136.5	136.5-152.5	Sand
173	Farmer City	(#7)	180	1967	1974	67	100	24	165	165-180	Sand
174	Heyworth	(#1)	61.7	1935	1935	18	380	4.2	41.7	41.7-61.7	Sand-Gravel
		(#2)	59	1959	1959	10	430	5	39	39-59	Sand-Gravel
175	Argenta	(#1)	230.5	1954	1954	65	174	11	219	219-230.5	Sand
		(#2)	251	1961	1961	63	178	18	229.7	229.7-251.2	Sand
176	Cisco	(#1)	113	1950	1950	45	80	32	105	105-113	Sand-Gravel
		(#2)	213	1958	1958	-	-	-	-	205-213	-
177	Deland	(#1)	83	1935	1935	18	65	46	73.5	73.5-83	Sand-Gravel
		(#3)	81	1952	1952	32	30	37	76	76-81	Sand-Gravel
		(#4)	79.5	1961	1961	31	32.5	39	76.2	76.2-79.5	Sand-Gravel
		(#5)	79	1961	1961	-	-	-	76	76-79	Sand-Gravel
178	Kenney		248	1956	1956	44	152	5.9	228	228-248	Sand-Gravel
179	LeRoy	(#1)	86	1918	1924	36	-	-	-	-	-
		(#2)	86	1918	1924	36	-	-	-	-	-

TABLE 2.4-33 (CONT'D)

WELL NUMBER	OWNER		DEPTH (ft)	DATE DRILLED	PUMP TEST DATA DATE	STATIC WATER LEVEL (ft)	PUMPING RATE (gpm)	DRAW-DOWN (FT)	CASING (ft)	PERFORATIONS (ft)	AQUIFER
		(#4)	79.5	1968 (deepened)	1968	46	150	10	67.5	67.5-79.5	Sand-Gravel
		(#5)	-	1943	-	-	-	-	-	-	-
		(#6)	102	1967	1967	44	300	16	87	87-102	Sand-Gravel
180	Maroa	(#1)	85	1892?	-	-	-	-	75	75-85	Sand-Gravel
		(#2)	292	1939	1976	112	130	8.6	270	270-292	Sand-Gravel
		(#3)	290	1948	1976	114	145	24	266	266-290	Sand-Gravel

NOTES:

1. The survey of non-private water wells located within a radius of 15 miles from the site was conducted initially by Dames & Moore for the PSAR. The survey was supplemented by Sargent & Lundy for the FSAR.

2. Data are from published and open file records of the Illinois State Water Survey.

3. Dashes indicated information not available.

4. Location of the wells are shown in Figure 2.4-45.

TABLE 2.4-34
PUBLIC WATER SUPPLY SYSTEMS WITHIN 15 MILES OF SITE

PUBLIC WATER SUPPLY SYSTEM	DISTANCE FROM STATION SITE (miles)	NUMBER OF PRODUCING WELLS	AQUIFER	AVERAGE DAILY USE (gpd/year)
Argenta	13.5	2	"Mahomet Sand"	69,000/1974
Cisco	13.0	1	Illinoian	45,000/1974
		1	Kansan	(Combined)
Clinton	7.1	7	"Mahomet Sand"	1,100,000/1974
DeLand	10.9	4	Illinoian	30,000/1974
DeWitt	0.9	1	"Mahomet Sand"	*,2
Farmer City	11.3	5	Illinoian or Kansan	213,000/1975
Heyworth	13.6	2	Alluvium	145,000/1976
Kenney	14.7	1	"Mahomet Sand"	28,000/1974
LeRoy	12.9	3	Wisconsinan	196,000/1973
Maroa	11.5	2	"Mahomet Sand"	120,000/1976
Wapella	7.9	2	Illinoian	50,000/1976
Weldon	5.8	2	Illinoian	30,000/1974
Weldon Springs	6.0	7	Wisconsinan	*
State Park		1 (spring)	and Illinoian	

NOTES:

1. Average daily use is given for the latest year for which data are available.

2. A small test well installed by Illinois Power Company in 1974 will be used by the village of DeWitt for future water supply. As of September 1979, installation of the distribution piping from the well to the village had not been completed.

* Pumpage data are not available.

TABLE 2.4-35
WATER WELLS WITHIN 5 MILES OF SITE

WELL NO.	OWNER	DEPTH (ft)	STATIC WATER LEVEL (ft)	YIELD gpm	YIELD DATE	DRAWDOWN (ft)	CASING (ft)	PERFORATIONS (ft)	AQUIFER
1*	Best	132	75	5	-	-	132	-	-
2	Best	75	40	10	-	-	75	-	-
3*	Alsup	196	80	10	-	-	196	-	-
4	Premer	97	50	10	-	-	97	-	-
5	Lisenby	87	35	-	-	-	87	-	-
6*	Palmer	337	-	0	-	-	-	-	-
7	Rudasill	70	40	-	-	-	70	-	-
8*	Lamkin	260	100	-	-	-	-	-	-
10*	Douglas	80	30	6	-	-	80	-	-
11	Mettler	101	39	-	-	-	-	-	-
12**	DeWitt School	156	54	-	-	-	78	-	-
13*	Spainhour	50	-	-	-	-	-	-	-
14*	Miller	212	-	-	-	-	-	-	-
15	Atteberry	59	10	-	-	-	-	-	-
16	Griffith	43	14	-	-	-	-	-	-
17	Robinson	35	18	-	-	-	-	-	-
18	Hess	40	20	-	-	-	-	-	-
19*	Camp Quest	120	82	-	-	-	-	-	-
20	Crawford	140	80	-	-	-		-	-
21*	Brighton	212	80	7	-	-	212	-	-
22	Unknown	-	-	-	-	-	-	-	-
23*	Unknown	20	18	5	1899	2	-	-	-
24	Querfeld	73	22	6	1932	8	70	70-73	Gravel
25	Atteberry	81	32	4	1944	34	-	-	Sand-Gravel
26	Webb	78	52	6	1946	3	74	74-78	Sand-Gravel
27*	Lane	47	30	3	1945	14	44	46-47	Sand
28*	Wantland	131	54	3	1947	72	-	-	Sand-Gravel
29	West	78	40	5	1939	7	75	75-78	Sand-Gravel
30	Wantland	61	35	5	1941	2	61	-	Gravel
31+	Unknown	226	94	4	1946	0	-	224-226	Sand-Gravel
32	Thompson	67	30	5	1945	4	67	65-67	Gravel
33	McCrossan	92	-	-	1936	-	-	-	-

TABLE 2.4-35 (CONT'D)

WELL NO.	OWNER	DEPTH (ft)	STATIC WATER LEVEL (ft)	YIELD gpm	YIELD DATE	DRAWDOWN (ft)	CASING (ft)	PERFORATIONS (ft)	AQUIFER
34	Sprague	70	20	6	1932	15	67	67-70	Gravel
35	Ives	#80	18	5	1940	12	77	77-80	Sand-Gravel
36*	Robison	87	57	5	1946	0	84	84-87	Sand-Gravel
37	Kauffman	50	16	5	1941	0	-	46-50	Sand-Gravel
38	Dinsmore	81	30	5	1927	20	-	-	-
39*	Stapleton	63	27	-	1946	24	-	63-63 1/2	Sand-Gravel
40	Shell Oil	228	114	35	-	0	-	132-228	Sand-Gravel
41	Reddick	72	50	6	1939	-	72	-	Gravel
42*	Spencer	41	17	3	1946	14	39	39-41	Coarse Sand
43*	Spencer	38	12	6	1945	8	35	34-37	Sand-Gravel
44*	Allen	115	50	5	1946	35	112	114-115	Gravel
45	Dawson	64	25	5	1939	10	64	-	Gravel
46	Callinson	72	25	6	1945	14	71	71-72	Gravel
47	Dyer	70	10	7	1939	2	67	64-70	Sand-Gravel
48	Swigart	92	29	5	1946	16	89	-	Sand-Gravel
49	Reeser	70	20	8	1938	5	70	-	Gravel
50*	Lane	250	99	6	1970	0	246	246-250	Sand
51	Cunningham	80	29	5	-	-	77	77-80	Gravel
52	Wantland	43	24	6	1940	1	40	-	Sand-Gravel
53	Foster	38	17	4	1968	8	-	36-38	Sand
54	Scott	46	17	4	1946	22	-	45-46	Sand-Gravel
55	Dinsmore	223	46	5	1940	2	180	-	Sand
56*	Feese	101	42	5	1940	6	101	59-62	Sand-Gravel
57	Wilson	184	90	5	1940	32	181	181-184	Sand-Gravel
58	Keys	128	50	6	1942	20	127	127-128	Sand
59	Tupy	52	21	6	1942	6	50	50-52	Gravel
60	Sasamon	180	100	20	1966	-	86	-	Rock
61	Dawson	228	70	7	1942	4	225	225-228	Sand
62	Gaby	74	25	5	1939	1	-	71-74	Sand-Gravel
63	Campbell	175	52	6	1942	6	172	173-175	Sand-Gravel
64	Thompson	22	10	5	1934	2	-	-	Gravel
65	Thompson	200	60		1928	-	-	-	Sand
66	Wilson	40	27	5	1916	8	-	-	Sand
67	Shue	62	32	5	1906	0	-	-	-
68	Orr	84	61	5	1900	0	-	-	-

TABLE 2.4-35 (CONT'D)

WELL NO.	OWNER	DEPTH (ft)	STATIC WATER LEVEL (ft)	YIELD gpm	YIELD DATE	DRAWDOWN (ft)	CASING (ft)	PERFORATIONS (ft)	AQUIFER
69	Shue	20	10	5	1922	5	-	-	Gravel
70	Dupree	23	7	5	1909	1	-	-	Sand
71	Dupree	16	12	5	-	4	-	-	Sand
72	Reynolds	30	24	5	-	2	-	-	Gravel
73	Fisher	75	15	5	1919	0	-	-	Sand
74	Griffith	7	4	5	1932	0	-	-	Sand
75	Giles	56	8	5	1904	0	-	-	Sand-Gravel
76	Lafferty	13	10	5	1919	0	-	-	Sand
77	Thord	52	32	4	1945	17	-	-	Sand-Gravel
78	Roben	25	19	5	1884	2	-	-	Sand-Gravel
79	Roben	35	10	-	1930	-	-	-	Sand-Gravel
80	Dinsmore	170	30	5	-	0	-	-	Gravel
81	Dinsmore	48	26	5	-	0	-	-	Coarse Gravel
82	Riggs	81	20	5	1929	0	-	-	Sand-Gravel
83	Swigart	30	15	5	-	15	-	-	Sand-Gravel
84*	Reeder	44	29	5	1910	6	-	-	Sand-Gravel
85*	Reynolds	98	48	5	1919	0	-	-	Sand-Gravel
86*	Ferguson	27	18	5	1896	9		-	Gravel
87*	Mettler	106	53	5	1927	53	-	-	Sand
88*	Burtoh	17	8	5	-	3	-	-	Gravel
89*	Burton	126	70	5	1929	0	-	-	Sand-Gravel
90	Torbert	58	20	3	1909	-	-	-	Gravel
91	Scott	35	15	3	1900	-	-	-	-
92	Y.M.C.A.	68	20	5	1909	0	-	-	Gravel
93	Sprague	34	16	3	1900	-	-	-	-
94	Luttrel	65	-	-	1916	-	-	-	Sand
95*	Allen	60	59	5	-	1	-	-	Gravel
96*	Benz	39	22	5	1913	17	-	-	Sand
97*	Reeder	30	15	5	-	15	-	-	Gravel
98	Shell Oil	115	60	-	-	-	-	-	Sand-Gravel
99*	Moore	18	10	5	1934	8	-	-	Sand-Gravel
100*	Reeser	18	12	5	-	2	-	-	Sand
101	Meltzer	38	12	5	1934	3	-	-	Gravel
102	Glenn	91	7	5	1933	0	-	-	Sand
103	Sommers	30	7	5	1934	3	-	-	Gravel

TABLE 2.4-35 (CONT'D)

WELL NO.	OWNER	DEPTH (ft)	STATIC WATER LEVEL (ft)	YIELD gpm	YIELD DATE	DRAWDOWN (ft)	CASING (ft)	PERFORATIONS (ft)	AQUIFER
104	Powers	75	10	5	1934	0	-	-	Gravel
105	Crawford	72	42	-	1955	-	-	-	Gravel
106	Watson	79	30	8	1955	0	-	-	Sand-Gravel
107	Larry	45	15	5	1934	0	-	-	Gravel
108	Reynolds	100	20	5	1943	0	-	49-56	Gravel
109	Maxwell	35	18	5	1934	12	-	-	Sand-Gravel
110	Kapp	57	10	5	1930	0	-	-	Sand-Gravel
111	Callison	88	4	5	1893	0	-	-	Sand
112	Crawford	18	8	5	1934	10	-	-	Gravel
113	Crawford	192	50	5	1933	0	-	-	Sand-Gravel
114	Morgan	30	15	5	1934	0	-	-	Gravel
115	White	65	20	5	1934	10	-	-	Gravel
116	Reeder	199	-	-	1934	-	-	-	Sand-Gravel
117	R.R. 10	176	49	-	1933	-	-	-	Sand-Gravel
118	Reeder	75	-	3	1953	-	-	-	Sand-Gravel
119	Moore	175	69	6	1943	1	-	173-175	Gravel
120**	Millikin Univ.	87	25	5	-	0	-	-	Sand
121	Blue	74	40	-	1940	-	-	-	Sand
122	Blue	72	25	5	1934	0	-	-	Gravel
123	Walters	70	35	5	1900	6	-	-	Sand-Gravel
124	Davis Girls Farms	70	20	5	1934	0	-	-	Sand
125*	Reeder	30	15	5	1934	2	-	-	Gravel
126*	Reeder	79	15	4	1953	25	-	77-79	Sand-Gravel
127	Campbell	40	15	5	1934	14	-	-	Sand
128	Reeser	30	20	5	1934	5			
155	Best	68	39	10	1971	3	64	64-68	Gravel
156	Payne	42	19	15	1969	0	38	38-42	Sand
157*	Palmer	150	-	-	-	-	-	-	Sand-Gravel
158	Tohill	71	-	Dry	1963	-	-	-	-
159	Spiddle	77		5	1947	11	73	73-77	Sand-Gravel
160	McConkey	52	21	6	1942	6	50	50-52	Gravel
161	Watson	72	25	6	1945	14	71	-	-
162*	Lane	20	Boring	-	1945	-	-	-	-

TABLE 2.4-35 (CONT'D)

WELL NO.	OWNER	DEPTH (ft)	STATIC WATER LEVEL (ft)	YIELD gpm	YIELD DATE	DRAWDOWN (ft)	CASING (ft)	PERFORATIONS (ft)	AQUIFER
163*	Lane	30	Boring	-	1945	-	-	-	-
164*	Lane	70	Boring	-	1945	-	-	-	-
165*	Lane	70	Boring	-	1945	-	-	-	-
166	Walker	70	20	8	938	5	67	67-70	Gravel
167	State Div. Highway	18	Boring	-	1937	-	-	-	-
168	Warner	60	20	10	1949	10	56	56-60	Sand-Gravel
169	Glenn	87	28	6	1940	12	-	84-87	Sand
*170**	DeWitt (Ill. Power Co.)	340	112	400	1974	60	300	300-340	Sand-Gravel
181*	Lane	60	Boring	-	-	-	-	-	-
182*	LaAne	69	Boring	-	-	-	-	-	-
183*	Lane	60	Boring	-	-	-	-	-	-
184*	Lane	60	Boring	-	-	-	-	-	-
185*	Lane	50	Boring	-	-	-	-	-	-
186*	Palmer	237	-	-	-	-	-	-	-
187*	Ferguson	107	-	-	-	-	-	-	-
188*	Best	128							
189*	Benz	149							
190*	Palmer	71							
191*	Miller	242	102	-	-	-	-	-	Sand
192*	Borpikus	264	-	-	-	-	-	-	-
193*	Borpikus	350	-	-	-	-	-	-	-
194	Lynn	80	-	-	-	-	-	-	Sand-Gravel
195	Lynn	35	-	-	-	-	-	-	Sand-Gravel
196	Lynn	35	-	-	-	-	-	-	Sand-Gravel
197*	Reinhart	247	-	-	-	-	-	-	Mahomet Sand- Gravel
198*	Reinhart	-	-	-	-	-	-	-	Sand-Gravel
199*	Reinhart	30	-	-	-	-	-	-	Sand-Gravel

NOTES:

1. The survey of private water wells located within a radius of 5 miles from the site was conducted by Dames & Moore for the PSAR.

TABLE 2.4-35 (CONT'D)

2. Data are from published records of the Illinois State Water Survey. Static water level, yield, and drawdown are from records of initial well testing.

3. Dashes indicate information not available.

4. Locations of the wells are shown in Figure 2.4-46.

* Well is on site property.

** Well supplies water for public use.

\+ Well located somewhere in Section 10, T19N, R3E; not shown in Figure 2.4-46.

TABLE 2.4-36
ADDITIONAL WATER LEVEL OBSERVATIONS

PIEZOMETER NUMBER	SURFACE ELEVATION (ft, MSL)	PIEZOMETER TIP ELEVATION (ft, MSL)	STRATIGRAPHIC UNIT OPEN TO PIEZOMETER	WATER LEVEL ELEVEVATION (ft, MSL)
P-31	736.8	577.8	Illinoian	711
P-36	738.2	515.2	Kansas	680
P-37	739.1	699.1	Wisconsinan	723
P-39	740.8	590.8	Illinoian	708
P-40	742.1	707.1	Wisconsinan	729

NOTES:

1. Locations of these piezometers are shown in Figure 2.5-16.

Table 2.4-37

TABULATION OF BOTTOM OF UHS ELEVATIONS

Homer L. Chastain & Associates Consulting Engineers			UHS Siltation Study Clinton Power Station	
Date: 1-20-80	Prep: JLR	Chkd: DS	Project: 2979	Sheet 1 of 1

Tabulation of Bottom of UHS Elevations

Point No.	Initial Elev. From Aerials	Elev. From 11-79 Survey	Elev. From 10-80 Survey	10-80 Elev. Minus Initial Elev.	10-80 Elev. Minus 11-79 Elev.
1	667.3	667.1	666.8	-0.7	-0.3
2	668.8	668.4	668.2	-0.6	-0.2
3	669.8	667.5	667.4	-2.4	-0.1
4	668.3	668.3	668.0	-0.3	-0.3
5	667.5	667.8	667.4	-0.1	-0.4
6	670.1	667.9	667.9	-2.2	0
7	668.0	667.8	667.9	-0.1	+0.1
8	667.9	667.0	667.1	-0.8	+0.1
9	668.0	668.0	667.1	-0.9	-0.9
10	667.6	668.3	668.4	+0.8	+0.1
11	668.2	668.0	668.4	+0.2	+0.4
12	667.8	667.6	667.8	0	+0.2
13	668.2	668.0	668.1	-0.1	+0.1
14	668.3	667.9	667.9	-0.4	0
15	668.0	667.9	668.0	0	+0.1
16	668.8	668.0	668.2	-0.6	+0.2
17	668.0	668.4	668.9	+0.9	+0.5
18	665.4	667.5	667.2	+1.8	-0.3
19	667.0	668.1	668.7	+1.7	+0.6
20	667.8	667.5	667.8	0	+0.3
21	668.0	667.4	667.7	-0.3	+0.3
22	664.8	665.8	665.9	+1.1	+0.1
23	666.7	667.3	667.6	+0.9	+0.3
24	667.4	667.9	668.4	+1.0	+0.5
25	667.5	668.0	668.3	+0.8	+0.3
26	667.7	668.2	667.9	+0.2	-0.3
27	667.6	668.0	668.0	+0.4	0
28	668.4	668.5	668.9	+0.5	+0.4
29	665.2	664.8	667.1	+1.9	+2.3
30	667.2	668.3	668.8	+1.6	+0.5
Ave. Elev.	667.7	667.7	667.9		

2.5 GEOLOGY, SEISMOLOGY, AND GEOTECHNICAL ENGINEERING

The Clinton Power Station site is located in the central stable region of North America in the Illinois Basin, slightly west of the La Salle Anticlinal Belt, approximately 6 miles east of the city of Clinton, DeWitt County, Illinois. The site area consists of a gently rolling upland developed on ground moraine, which has been dissected by the southwest-flowing Salt Creek and the North Fork of Salt Creek. Topographic relief varies from approximately 10 feet on the upland to a maximum of about 80 feet between the upland and valley bottoms. Strata underlying the site consists of an estimated 170 to 360 feet of Quaternary overburden, largely Wisconsinan, Illinoian, and pre-Illinoian aged glacial deposits resting on essentially flat-lying Pennsylvanian-aged shales, sandstones, and thin coal beds.

A horizontal ground surface acceleration of 25% of gravity (0.25g) applied at foundation level was selected for the safe shutdown earthquake (SSE). This acceleration value was derived by assuming an Intensity VIII event (MM) centered near the site.

Extensive geotechnical investigations carried out prior to and during construction (including geologic mapping of the excavations) showed nothing that would preclude safe construction or operation of a nuclear-fueled power station. There are no known faults or folds of design significance at or anywhere near the site.

Major power block structures were constructed on mat foundations underlain by compacted fill resting on hard Illinoian till. The cooling lake was formed by construction of an earth-filled dam across Salt Creek downstream of its confluence with the North Fork of Salt Creek. The emergency cooling ultimate heat sink was formed by constructing a submerged pond within the cooling lake reservoir in the North Fork of Salt Creek. Required pond capacity was developed by construction of an earth dam across the natural stream channel and excavation upstream from the berm. Adequate borrow was available for all fills. Suitable founding strata were available for all structures.

Geologic investigations undertaken include: a review of published and unpublished data; discussions with individuals, agencies, and companies having information in the region or site area; reconnaissance field investigations, drilling and sampling, and surface and borehole geophysics; laboratory testing of soil, rock, and water samples, and detailed geologic mapping of excavations. The firms who performed the work are as follows:

INVESTIGATIONS	PERFORMED BY
Geologic Literature Review	Dames & Moore, and Sargent & Lundy
Geologic Reconnaissance	Dames & Moore
Geologic Mapping of Excavations	Sargent & Lundy
Test Borings	Dames & Moore, and Sargent & Lundy
Geophysical Explorations	Dames & Moore

Laboratory Tests	Dames & Moore, U.S. Testing, Westenhoff & Novick, and Soil Testing Services Inc.
Foundation Considerations	
Subsurface Conditions	Dames & Moore, and Sargent & Lundy
Foundations	Sargent & Lundy
Vibratory Ground Motion	Dames & Moore, and Sargent & Lundy
Surface Faulting	Dames & Moore, and Sargent & Lundy
Stability of Subsurface Materials	Dames & Moore, Sargent & Lundy, and Woodward-Clyde Consultants
Stability of Slopes	Sargent & Lundy
Embankments and Dams	Sargent & Lundy
Groundwater	Dames & Moore, and Sargent & Lundy

2.5.1 Basic Geologic and Seismic Information

2.5.1.1 Regional Geology

The region surrounding the site lies within the Central Stable Region of the North American Continent (Reference 1). This province is a tectonically stable area characterized by gently dipping sedimentary rock of Paleozoic overlain by thin Cenozoic deposits mostly quarternary glacial drift, and, locally by Mesozoic strata. Beneath the Paleozoic is a basement complex of Precambrian and igneous and metamorphic rocks. Intermittent slow subsidence and gentle uplift through the Paleozoic has resulted in broad basins (e.g., the Illinois, Michigan, and Forest City Basins), filled with gently dipping sedimentary rocks, and in intervening broad arches or highs (e.g., the Kankakee Arch, Mississippi River Arch, etc.). Locally, folds and faults have been superimposed on this pattern. The Clinton site is located on the northwest flank of the Illinois Basin, west of the La Salle Anticlinal Belt.

The Paleozoic sedimentary rock sequence is punctuated by several unconformities of regional importance, reflecting widespread advances and withdrawals of the Paleozoic seas across the interior of North America.

Locally, in the regional area, there was Cretaceous and Tertiary deposition in a few local areas in the Mississippi Embayment, western Illinois, eastern Missouri, and southern Indiana. During Quaternary time, widespread deposition occurred in the regional area as the result of continental glaciation. Approximately 170 to 360 feet of Quaternary deposits overlie the Pennsylvanian bedrock at the Clinton site. These deposits consist of a complex sequence of glacial drift, stream alluvium, and loess.

2.5.1.1.1 Regional Physiography

The region of the United States in which the Clinton Power Station site is located is part of the Till Plains Section of the Central Lowland Physiographic Province (Reference 2). The terrain aspect of central Illinois and adjacent Indiana is typical of the province, consisting of undulating, low-relief topography formed by a glacial drift cover whose thickness ranges from a few tens of feet to several hundreds of feet. Much of the Till Plains Section is characterized by landforms of low, commonly arcuate ridges, called moraines, interspersed with relatively flat intermorainal areas. The Clinton Power Station site is situated in a sector of the Till Plains Section known as the Bloomington Ridged Plain as shown in Figure 2.5-1.

Postglacial stream development has dissected the drift mantle and in some areas along the main valleys, preglacial bedrock has been exposed by erosion; however, there are no bedrock exposures near the site area.

Elevations on the general drift surface between drainageways in the general area of the site average about 740 feet above sea level.

2.5.1.1.2 Regional Stratigraphy

Overburden deposits consisting of Quaternary-aged glacial drift and stream alluvium overlie thick sequences of Paleozoic sedimentary rock throughout most of Illinois and adjacent Indiana. In the extreme northern part of Illinois, the drift rests principally upon Ordovician and Silurian formations. Elsewhere, the uppermost strata beneath the glacial drift consist mainly of Pennsylvanian-aged (Late Paleozoic) rocks. Figure 2.5-2 illustrates the general rock sequence in central Illinois; and Figure 2.5-3 shows the regional bedrock geology of Illinois and surrounding states. Most of the Paleozoic formations in Illinois dip gently (about 25 feet per mile) with some thickening toward the axis of the Illinois Basin in southeastern Illinois. Figure 2.5-4 shows the regional stratigraphic relationships along north-south and east-west cross sections near the site. The ages for the geologic periods discussed below are taken from Faul (Reference 3).

2.5.1.1.2.1 Cenozoic Era (Present to 65 ± 2 million years B.P.)

2.5.1.1.2.1.1 Quaternary System (Present to 2 ± 1 million years B.P.)

The surficial deposits of most of the regional area are Quaternary in age and are classified as part of the Pleistocene Series. The deposits consist predominantly of glacial or glacially-derived sediments of glacial till, outwash, loess (a wind-blown silt), and glaciolacustrine deposits, as well as alluvium. The age and geomorphic relations of the glacial deposits in northern and central Illinois are shown in Figure 2.5-1. The stratigraphic sequence of Pleistocene deposits in Illinois is shown in Figure 2.5-5.

There were four major periods of glaciation during Pleistocene time in the regional area. From oldest to youngest, these periods are known as the Nebraskan, Kansan, Illinoian, and Wisconsinan Stages. (The present classification of pre-Illinoian Drift, however, has been questioned by some, see Reference 4.) During each of these glacial periods, glaciers periodically advanced and retreated across parts of the regional area. Consequently, a complex sequence of deposits developed, as evidenced by the stratigraphic classification for the Pleistocene deposits of Illinois (Figure 2.5-5).

Nebraskan and Kansan age glacial deposits are present at the surface and in the subsurface of the regional area in Iowa, Missouri, and parts of western and east-central Illinois.

Illinoian age deposits are present beyond the limit of Wisconsinan glaciation in northern and central Illinois (Figure 2.5-1). Illinoian age deposits are also found beneath the Wisconsinan drift cover up to 20 to 40 miles back from the Wisconsinan front (Reference 2). The site is located a few miles inside the limit of Wisconsinan glaciation (Figure 2.5-1).

Quaternary deposits in the regional area locally exceed 400 feet in thickness but are generally much thinner. They cover an irregular bedrock surface, largely erosional in origin characterized by valleys and uplands that were developed before and during glacial time (Reference 6). The locations of buried bedrock valleys in Illinois are shown in Figures 2.5-6 and 2.5-7.

The sequence of Pleistocene deposits in the regional area is comprised of glacial or glacially-derived sediments of glacial till, outwash, loess, and glaciolacustrine deposits. Glacial till is a poorly sorted sediment consisting of a matrix of sand, silt, and clay with interspersed pebbles or cobbles. Individual glacial till units tend to be texturally and mineralogically distinctive and uniform, and can be traced over wide areas (References 5 and 7). Glacial till can be deposited by a number of subglacial and supraglacial processes. Discontinuous lenses of stratified sand, silt, or gravel may occur within glacial till and are associated with melt waters from glacial ice (Reference 8).

Outwash deposits consist dominantly of sand and gravel with minor amounts of silt and clay. The texture of outwash deposits can vary greatly both laterally and vertically due to differences in the characteristics of the meltwater channels, and differences in meltwater velocity caused by diurnal and seasonal differences in the rate of meltwater discharge.

Loess is a wind-blown silt derived principally from the fine-grained sediments along meltwater rivers and channels. Loess thickness decreases eastward away from the major rivers in the regional area which served as major drainageways for meltwater streams. In the bluffs bordering the eastern side of major river valleys, loess typically is on the order of a few tens of feet in thickness thinning to only a few feet in thickness a short distance away from the river bluffs.

Glaciolacustrine deposits were formed in lakes, on or fed by glaciers. Glaciolacustrine deposits are predominantly silts and clays, although they may locally be sands or gravels near the margins where they were parts of beaches, bars, or deltas.

Between glacial periods, the climate returned to more temperate conditions. As the glacial and glacially-derived sediments were exposed, weathering processes began to alter them, and soils were formed. The thickness and character of the resulting soils are largely a function of climate, topographic position, vegetation, and duration of the interglacial stage.

The youngest deposits in the regional area are classified as Holocene in age (Reference 2). They consist of alluvium formed along present rivers and drainageways, colluvium, dune sand, peat, and lacustrine deposits formed in present lakes.

2.5.1.1.2.1.2 Tertiary System (2 ± 1 to 65 ± 2 million years B.P.)

Late Tertiary deposits consist of chert gravels in the Mississippi Embaymment, of high-level, Plio-Pleistocene isolated patches of chert gravel in western Illinois (Reference 2) and eastern Missouri (Reference 9), and of Mio-Pliocene chert gravels in southern Indiana (Reference 10.)

2.5.1.1.2.2 Mesozoic Era (65 ± 2 to 225 ± 5 million years B.P.)

2.5.1.1.2.2.1 Cretaceous System (65 ± 2 to 135 ± 5 million years B.P.)

Cretaceous age sediments in the regional area are present only in a small area in western Illinois and in the Mississippi Embayment area of extreme southern Illinois (Figure 2.5-3), and are Late Cretaceous in age. In western Illinois the Cretaceous age rocks are up to 100 feet thick and consist of a basal gravel overlain by sands and clayey sands. These strata apparently are the easternmost outliers of Cretaceous sediments that formerly covered the region east of the Rocky Mountains and north of the Ozarks (Reference 2).

In southern Illinois, the Cretaceous age rocks of the Mississippi Embayment area consist of up to 500 feet of chert gravels, deltaic sands, and marine clays (Reference 2).

2.5.1.1.2.2.2 Jurassic System (135 ± 5 to 190 ± 5 million years B.P.)

There are no known deposits of Jurassic age in the regional area.

2.5.1.1.2.2.3 Triassic System (190 ± 5 to 225 ± 5 million years B.P.)

There are no known deposits of Triassic age in the regional area.

2.5.1.1.2.3 Paleozoic Era (225 ± 5 to approximately 600 million years B.P.)

2.5.1.1.2.3.1 Permian System (225 ± 5 to 270 ± 5 million years B.P.)

There are no known deposits of Permian age in the regional area.

2.5.1.1.2.3.2 Pennsylvanian System (270 ± 5 to 320 ± 10 million years B.P.)

The bedrock surface throughout much of Illinois has been developed on strata of Pennsylvanian age, as shown in Figure 2.5-3. This surface is covered by Pleistocene drift except where erosion has removed these glacial materials. The base of the Pennsylvanian is an unconformity of major regional extent.

The Pennsylvanian sequence is composed of cyclothem units formed by alternating beds of shale, sandstone, coal, limestone, and siltstone. The thickness of the Pennsylvanian age rocks varies greatly depending upon the amount of subsequent erosion, but it is generally on the order of a few hundred feet to 700 or 800 feet in the site region. Further south, the Pennsylvanian in the Illinois Basin thickens to more than 2200 feet.

2.5.1.1.2.3.3 Mississippian System (320 ± 10 to 340 ± 10 million years B.P.)

Strata of the Mississippian System outcrop along the outer margins of the Illinois Basin (Figure 2.5-3), and are present in the subsurface of the basin as well. Mississippian-age sediments consist predominantly of limestone, with lesser siltstone and shale, and are on the order of 500 to 600 feet in thickness near the site area.

2.5.1.1.2.3.4 Devonian System (340 ± 10 to 400 ± 10 million years B.P.)

Devonian age rocks, consisting predominantly of shale and limestone, underlie much of the general region. These sediments are on the order of about 200 feet thick near the area of the site. There is a major unconformity at the base of Middle-Devonian strata.

2.5.1.1.2.3.5 Silurian System (400 ± 10 to 430 ± 10 million years B.P.)

Silurian age rocks underlie the Devonian strata in the general region and outcrop over large areas of northern Illinois and Indiana beneath a cover of glacial drift. Near the site area Silurian strata form the bedrock surface in parts of the La Salle Anticlinal Belt. The Silurian rocks are dominantly carbonates, some of which include reef structures. Silurian rock in the site area is approximately 450 feet thick (Reference 2).

2.5.1.1.2.3.6 Ordovician System (430 ± 10 to approximately 500 million years B.P.)

Ordovician age rocks outcrop in the northern, southwestern, and southeastern parts of the regional area (Figure 2.5-3), and are present in the subsurface of most of the rest of the regional area. The Ordovician has been divided into three series, which from oldest to youngest are the Canadian, Champlainian, and Cincinnatian Series. The Canadian Series (Lower Ordovician) is represented by dolomite and sandstone. A major erosional unconformity separates the top of the Canadian Series from the basal sandstone of the overlying Champlainian Series. Strata of the Champlainian Series include the basal sandstone (St. Peter Sandstone) and an overlying sequence of limestone and dolomite. Upper Ordovician rocks of the Cincinnatian Series are dominantly shales. Ordovician rocks in the site area are approximately 1500 feet thick (Reference 2).

2.5.1.1.2.3.7 Cambrian System (Approximately 500 to 600 million years B.P.)

The Cambrian in Illinois and adjoining states is represented by only the Upper Cambrian Croixan Series, a sequence of sedimentary rocks ranging from 1500 to over 3500 feet in thickness in Illinois (Reference 2). Approximately the lower two-thirds of the Croixan Series is sandstone in northern and central Illinois, grading to dolomite further south. The upper one-third of the Croixan Series consists of interbedded siltstone, shale, sandstone, and dolomite. Cambrian strata rest on the Precambrian basement with profound unconformity. Approximately 3100 feet of Cambrian rocks underlie the site (Reference 2).

2.5.1.1.2.4 Precambrian Era (Greater than approximately 600 million years B.P.)

Precambrian igneous rocks, approximately 1.1 to 1.4 billion years in age lie below the surface throughout the regional area. Precambrian rocks are at the surface in the Ozarks, more than 185 miles southwest of the site (Figure 2.5-3), but are generally deeply buried in the regional area, ranging in depth from 2000 feet near the Illinois-Wisconsin border to over 13,000 feet in the deepest part of the Illinois Basin in southeastern Illinois. Precambrian rocks in the regional area are dominantly granite with associated granodiorite, rhyolite, felsite, or granophyre of closely related composition (Reference 2).

2.5.1.1.3 Historical Geology

The geologic history of the regional area is discussed by geologic eras, which are subdivided into periods. The strata formed during the periods are classified as time-rock units and are designated as systems. In the following discussions, the periods are divided into early, middle, and late; the corresponding strata are designated as series and referred to as lower, middle, and upper, respectively.

Regional stratigraphy is discussed in Subsection 2.5.1.1.2, and regional structural geology is discussed in Subsection 2.5.1.1.4.

The ages given represent the broad time spans and are not restricted to those portions of the time interval represented by the rocks within the regional area.

2.5.1.1.3.1 Precambrian Era (Greater than approximately 600 million years B.P.)

Except for a few isolated exposures in the Ozarks, the Precambrian basement complex within the regional area is covered by younger strata. Outcrop and subsurface data on the Precambrian rocks indicate that the Precambrian basement in the regional area consists of igneous rocks, chiefly granite and associated rocks, that formed approximately 1.4 to 1.1 billion and in part 0.64 billion years ago (Reference 2). By their ages, the Precambrian rocks are assigned to the Precambrian Y division of the U.S. Geological Survey (Reference 11). A period of erosion, lasting 600 to 900 million years (Reference 2), followed the formation of the Precambrian basement, and resulted in up to 2000 feet of relief on the Precambrian surface in the Ozarks (Reference 12), and several hundred feet of relief elsewhere in the regional area (Reference 2).

2.5.1.1.3.2 Paleozoic Era (Approximately 600 to 225 ± 5 million years B.P.)

2.5.1.1.3.2.1 Cambrian Period (Approximately 600 to 500 million years B.P.)

Within the regional area, deposition of sediments on the Precambrian basement did not begin until Late Cambrian time when shallow seas transgressed from the south. Initial sediments in the regional area consisted of a sequence of sandstones (Reference 2). These basal Cambrian clastics (Potsdam Sandstone Megagroup) grade southward and upward into carbonates.

2.5.1.1.3.2.2 Ordovician Period (Approximately 500 to 430 ± 10 million years B.P.)

Sedimentation in the regional area continued from Late Cambrian into Early Ordovician time with no appreciable break. Early Ordovician sediments in the regional area are dominantly limestones and dolomites with some sandstones (Reference 2).

The Reelfoot Basin in western Kentucky, outside of the regional area, appears to have been the center of thickest deposition in Early Ordovician time (Reference 13).

Regional uplift occurred in the interval between Early and Middle Ordovician time, resulting in a major unconformity between rocks of Early and Middle Ordovician age throughout much of the central interior region, and in the development of karst in Illinois (Reference 14). During this interval, the Wisconsin Dome was uplifted, and the Kankakee Arch began to form, separating the Illinois Basin from the Michigan Basin (Reference 14). The Kankakee Arch at this time was at a position somewhat northeast of its later location (Reference 14). Uplift also occurred in the southwestern part of the regional area in the Ozarks.

In Middle Ordovician time, shallow seas readvanced into the area. The initial deposits were sandstone, followed by an extensive sequence of carbonates.

In the interval between Middle and Late Ordovician time, the shallow seas retreated and then readvanced back into the regional area, as evidenced by the unconformity separating rocks of Middle and Late Ordovician age. Late Ordovician rocks in the regional area are dominantly shales, possibly reflecting uplift in the Appalachian region during the Taconic orogeny (Reference 14). The center of thickest deposition in the regional area migrated north during Ordovician time from the Reelfoot Basin, south of the regional area, to the southern Illinois area of the Illinois Basin and the Fairfield Basin (Reference 13).

2.5.1.1.3.2.3 Silurian Period (430 ± 10 to 400 ± 10 million years B.P.)

Rocks of Silurian age in the regional area were deposited on an erosional unconformity of low relief (Reference 14), which indicates a period of sea withdrawal and readvance in the interval from the end of Ordovician sedimentation to the beginning of Silurian sedimentation. The lower Silurian Series are predominantly carbonates, whereas Middle and Late Silurian strata are characterized by limestones or dolomites containing organic reefs and bioherms.

Initial upwarp of the Sangamon Arch (see Subsection 2.5.1.1.4.1.13) may have occurred in Late Silurian time (Reference 14). The Wisconsin Dome underwent a second period of significant uplift after deposition of Silurian strata (Reference 15). The Lincoln Anticline also began to develop in Silurian or Devonian time (Reference 16).

2.5.1.1.3.2.4 Devonian Period (400 ± 10 to 340 ± 10 million years B.P.)

Early Devonian sedimentation was restricted to deposition of limestones and cherts in the southern part of the Illinois Basin.

A major unconformity occurs at the base of the Middle Devonian in the regional area. This extensive unconformity is the result of regional uplift with withdrawal of the seas. The Ozark area was again uplifted during this time (Reference 17).

Shallow seas readvanced into the regional area, and early-Middle Devonian sediments in the Illinois Basin were primarily limestones. In the late-Middle and Late Devonian, organic shale (New Albany Shale Group), derived from the Acadian Highland to the east, were deposited in the Illinois Basin (Reference 14).

2.5.1.1.3.2.5 Mississippian Period (340 ± 10 to 320 ± 10 million years B.P.)

Sedimentation in the Illinois Basin was generally continuous from Devonian into Mississippian time. By Middle Mississippian time, sedimentation conditions in the regional area had changed from dominantly clastic deposition to the accumulation of limestones and dolomites. Upper Mississippian sedimentation consisted of alternating deposits of clastics and carbonates. The main source of the Upper Mississippian age clastic sediments appear to have been from the northeast, although some sediment came from the northwest and from the Ozark region (Reference 2).

The principal folding along the Cap au Gres Faulted Flexure was post-Middle Mississippian and pre-Pennsylvanian, although minor movements occurred before and after this period.

Development of the Mississippi River Arch began in Mississippian time and continued into Pennsylvanian time (Reference 14).

2.5.1.1.3.2.6 Pennsylvanian Period (320 ± 10 to 270 ± 5 million years B.P.)

A major unconformity of regional dimension separates rocks of the Mississippian and Pennsylvanian Series in the regional area. Strata in the regional area were warped, faulted, and truncated by erosion at the close of Mississippian time. Valleys as much as 450 feet deep were cut into Mississippian age strata and were subsequently filled with Pennsylvanian sediments (Reference 2). During the period marked by the Mississippian-Pennsylvanian unconformity, the La Salle Anticlinal Belt began to develop. As the La Salle Anticlinal Belt rose, the locus of maximum deformation moved progressively southward from the La Salle area (Reference 14). Renewed uplift also occurred in the Ozark area at this time, causing widespread erosion (Reference 17).

Pennsylvanian sediments in the regional area were deposited in a gently subsiding trough, the Illinois Basin, that was open toward the south until post-Pennsylvanian time, when it was closed by uplift of the Pascola Arch, south of the regional area (Reference 2). The depositional environments during Pennsylvanian time were generally different than those of earlier Paleozoic time, and resulted in delta plain, brackish water, and marine sediments. Much of the sedimentation occurred in large deltas on the gently subsiding basin. Ninety to ninety-five percent of the Pennsylvanian strata consist of clastic rocks (Reference 2). Pennsylvanian age strata are characterized by vertical changes in lithology, commonly abrupt. In Illinois, 500 or more distinguishable units of sandstone, siltstone, shale, limestone, coal, and underclay grouped into cyclotherms can be distinguished (Reference 2).

Pennsylvanian deposits thin over the La Salle Anticlinal Belt indicating continued uplift along this structure during Pennsylvanian time. The last period of folding along the La Salle Anticlinal Belt took place in post-Pennsylvanian time (Reference 18).

Renewed uplift occurred in the Ozark area in post-Pennsylvanian time. This was followed by erosion over the entire state of Missouri (Reference 17).

2.5.1.1.3.2.7 Permian Period (270 ± 5 to 225 ± 5 million years B.P.)

No deposits of Permian age are present in the regional area. Permian sediments may have been deposited in the regional area (References 2 and 14), but if so, they have subsequently been removed by erosion.

2.5.1.1.3.3 Mesozoic Era (225 ± 5 to 65 ± 2 million years B.P.)

2.5.1.1.3.3.1 Triassic Period (225 ± 5 to 190 ± 5 million years B.P.)

There are no deposits of Triassic age in the regional area. This was largely a period of erosion in the regional area (Reference 2).

2.5.1.1.3.3.2 Jurassic Period (190 ± 5 to 135 ± 5 million years B.P.)

There are no deposits of Jurassic age in the regional area. This was largely a period of erosion in the regional area (Reference 2).

2.5.1.1.3.3.3 Cretaceous Period (135 ± 5 to 65 ± 2 million years B.P.)

Beginning in Late Cretaceous time, shallow seas once again advanced into parts of the regional area, leaving beach, nearshore, or deltaic deposits in portions of southern and western Illinois (Figure 2.5-3). The Late Cretaceous age rocks in western Illinois are apparently the easternmost outliers of Cretaceous sediments that formerly covered the region east of the Rocky Mountains and north of the Ozarks. The sediments are characteristic of beach and nearshore deposits (Reference 2).

The Late Cretaceous age rocks of extreme southern Illinois were part of a large delta formed where a river from the east discharged into the Mississippi Embayment (Reference 2).

2.5.1.1.3.4 Cenozoic Era (65 ± 2 million years B.P. to the present)

2.5.1.1.3.4.1 Tertiary Period (65 ± 2 to 2 ± 1 million years B.P.)

There are no deposits of the Paleocene, Eocene, Oligocene, or Miocene epochs present in the regional area except for gravels in Indiana of possible Mio-Pliocene age (Reference 10). This was likely a period of erosion. Late Tertiary gravels in part of the Pliocene Series are present in the regional area in widely scattered outcrops (Reference 2). The Pliocene was primarily a time of erosion with some local areas of fluvial deposition. This deposition is represented by relict patches of chert and quartz gravel, part of which may be reworked from older Tertiary or Cretaceous gravels. Reworking of these Pliocene gravels may have continued into early Pleistocene (Reference 2).

2.5.1.1.3.4.2 Quaternary Period (2 ± 1 million years B.P. to the present)

Most of the regional area was covered by continental ice sheets during Pleistocene time. Prior to the onset of Pleistocene glaciation, Tertiary erosion cycles had left the Paleozoic sediments as an essentially planar surface dissected by stream valleys (Reference 2). Advances of the continental ice sheets during substages within the Nebraskan (oldest), Kansan, Illionian, and Wisconsinian (youngest) stages have left a complex sequence of deposits.

The glacial stages were separated by interglacial periods. Soils formed during these periods are generally well known and widely utilized stratigraphic horizons.

Glaciations also altered the topography throughout much of the region and rearranged patterns (including the Mississippi River).

Since the disappearance of the last of the Pleistocene glaciers, the region has been largely undergoing a period of erosion and most likely isostatic rebound.

The youngest deposits in the regional area are classified as Holocene in age (Reference 2). They consist of alluvium formed along present rivers and drainageways, colluvium, dune sand, peat, and lacustrine deposits formed in present lakes.

2.5.1.1.4 Regional Structural Geology

The dominant bedrock structural features within the study region are illustrated in Figures 2.5-8, 2.5-9 and 2.5-10. An attempt has been made to show as many structural features within the study region as possible; however, some generalization has been necessitated by the small-scale mapping of closely spaced features such as zones of intense faulting in southern Illinois.

Data tabulations for all important structures within a 200-mile radius are given in Tables 2.5-1 and 2.5-2 (see also Attachment D2.5).

The dominant structures of the regional area and vicinity are the Illinois Basin and its bounding structures: the Mississippi River Arch, Wisconsin Arch, Kankakee Arch, Cincinnati Arch, Mississippi Embayment, Pascola Arch, Ste. Genevieve Fault Zone, and Lincoln Anticline. Within or crossing the Illinois Basin are major structures; such as the Sandwich Fault Zone, La Salle Anticlinal Belt, Rough Creek Fault Zone, and Wabash Valley Fault System; and structures of relatively less significance, such as the DuQuoin Monocline and Louden Anticline, for example. On the bounding arches are structures such as the Royal Center Fault and Fortville Fault. The movement on structures in the regional area was intermittent, and confined essentially to the Paleozoic.

2.5.1.1.4.1 Folding

The site occupies the sediment-covered part of the craton near the interior part of the Illinois Basin and lies some 15 to 20 miles west of the La Salle Anticlinal Belt (see Figure 2.5-8).

The distribution of major folds in the region is shown in Figure 2.5-9 and their characteristics are presented in Table 2.5-1. The knowledge of these structural features is based on surface and/or subsurface geological data. Some of the structures in the regional area have experienced post-Pennsylvanian movement. Due to the absence of sediments representing the interval from Pennsylvanian to Cretaceous or Pleistocene time, the age of final movement on these structures cannot be precisely dated. However, based on stratigraphic relationships and geologic history outside of the regional area, it is presumed that most of the post-Pennsylvanian tectonic activity is related to Appalachian structural development that occurred near the close of the Paleozoic Era (Reference 14).

The direction and amount of regional dip of the strata in northern and central Illinois vary. In the vicinity of the project area, the dip is gently southwest on the flank of the Downs Anticline at about 25 to 30 feet per mile.

2.5.1.1.4.1.1 Illinois Basin

The Illinois Basin is an oval-shaped basin with the major axis trending approximately N 25° W. The major axis of the basin is approximately 350 miles long, and the minor axis is approximately 250 miles long (see Figures 2.5-8 and 2.5-9). The deepest part of the basin, the Fairfield Basin, is in southeastern Illinois. Sediments in the Fairfield Basin are 12,000 to 14,000 feet thick (Reference 19).

In a purely structural sense, the Illinois Basin could be said to extend out to the axes or crests of the bounding arches (Reference 20). Strata in the Illinois Basin rises gently north at an average of 1° or less 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. To the south, the Illinois Basin rises to the Pascola Arch (which is outside of the regional area). To the southwest, the Illinois Basin is bordered by the Ste. Genevieve Fault Zone. To the west and northwest, the Illinois Basin is bordered by the Lincoln Anticline and the Mississippi River Arch. The site is located in the central portion of the Illinois Basin, north of the area of greatest structural depression.

The Illinois Basin began to form in Cambrian time and continued to develop intermittently until the end of the Paleozoic (Reference 2). The center of thickest deposition of the basin migrated northward during the Paleozoic (Reference 13).

2.5.1.1.4.1.2 Wisconsin Arch and Kankakee Arch

The Wisconsin Arch is a south-to southeast-trending extension of the Wisconsin Dome. 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 (Reference 15). 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 it.

2.5.1.1.4.1.3 La Salle Anticlinal Belt

The La Salle Anticlinal Belt is more than 200 miles long and extends from a point north of the Illinois River, near La Salle, to the Indiana State line on the Wabash River south of Vincennes. Its closest approach is 15 to 20 miles to the east of the site. A portion of the La Salle Anticlinal Belt is reflected in the regional bedrock geology map as a finger of rocks of varied ages, from Silurian through Mississippian, flanked by younger Pennsylvanian units (see Figure 2.5-3). This feature is shown on the regional geologic sections (Figure 2.5-4).

The La Salle Anticlinal Belt is a complex structure consisting in many places of en echelon north-south trending folds and troughs (Reference 18). Structures of the La Salle Anticlinal Belt are commonly asymmetrically folded to the west and are nearly monoclinal. Dips on the west flank of the belt may be up to 2000 feet per mile (approximately 20°), while the eastern flank is generally much more gently dipping, 25 to 50 feet per mile (approximately 1/2°) (Reference 21). Several en echelon folds and troughs of the La Salle Anticlinal Belt within 50 miles of the site have been named. To the north and east some 5 to 10 miles from the site, is a small flexure trending parallel to the La Salle Anticlinal Belt known as the Downs Anticline. Several domes are present along this structure and lie to the north and east of the site. These features are discussed further in Subsection 2.5.1.2.3.

A series of subsidiary structures of the La Salle Anticlinal Belt lie 15 to 35 miles southeast of the station site. These structures, beginning with the most westerly, are the Mattoon and Tuscola Anticlines and the Murdock and Marshall Synclines (Figure 2.5-9). They are all minor structures associated with the La Salle Anticlinal Belt within the Illinois Basin and they all trend nearly parallel to the La Salle Anticlinal Belt. The Mattoon Anticline is a small positive structure. The Tuscola Anticline plunges southeastward and its west limb forms the west limb of the La Salle Anticlinal Belt. The Murdock Syncline plunges gently southward and is adjacent to the east limb of the Tuscola Anticline. The Marshall Syncline is asymmetrical, with a steep west flank, and plunges gently southward.

The Ashton Arch (Figures 2.5-8 and 2.5-9) may be an extension of the La Salle Anticlinal Belt.

Initial deformation along the La Salle Anticlinal Belt took place in Mississippian or pre-Mississippian time. Deformation probably continued intermittently until the close of Paleozoic time (Reference 22).

2.5.1.1.4.1.4 Mississippi River Arch

The Mississippi River Arch is a broad, corrugated fold which parallels the Mississippi River, with contiguous parts in Illinois, Iowa, and Missouri. It is located approximately 120 miles west of the site (Figures 2.5-8 and 2.5-9). The western flank of the arch gently subsides into the Forest City Basin (Reference 17). Development of the arch began in Mississippian time and continued into the Pennsylvanian, as indicated by the thinning of sedimentary strata which rise onto the arch from adjoining basins (Reference 14). The arch was probably subjected to additional deformation at the close of the Paleozoic (Reference 22).

2.5.1.1.4.1.5 Lincoln Anticline

The Lincoln Anticline trends approximately northwest-southeast for 165 miles through eastern Missouri and western Illinois (Figures 2.5-8 and 2.5-9), and separates the Forest City Basin (which lies outside of the regional area) from the Illinois Basin. The fold is not a simple anticlinal structure, but rather a regional uplift upon which are superimposed domes, anticlines, synclines, and faults. The structure is asymmetric and has a maximum structural relief of 1000 feet (Reference 17). The fold is bounded on the south by the Cap Au Gres Faulted Flexure. The southwest side of the fold is marked by comparatively steep dips and faulting, while the northeast flank is marked by gentle dips and the absence of faulting. The Lincoln Anticline is believed to have formed intermittently from Ordovician to post-Pennsylvanian time (References 16, 23). The main period of development of the fold was in the interval from Late Mississippian to Early Pennsylvanian time (Reference 23).

2.5.1.1.4.1.6 Ozark Uplift

The central part of the Ozark Uplift or Dome is located approximately 200 miles southwest of the site. The Ozark Uplift is the major structural feature in Missouri, and is a broad, slightly asymmetrical, quaquaversal fold (Reference 17). The structural center of this uplift is in Iron County, Missouri. The topographic axis extends from Barry to Iron Counties. The boundaries of the Ozark Uplift are somewhat vague in areas, particularly to the north and northwest; however, they generally correspond to the Ordovician-Mississippian rock contacts to the east and west and to the Mississippi Embayment to the south. The Ozark Uplift is separated from the Illinois Basin by the Ste. Genevieve Fault Zone. The Ozark uplift underwent uplift intermittently in Paleozoic, Mesozoic, and Tertiary time and perhaps into the present as well (Reference 17).

2.5.1.1.4.1.7 DuQuoin Monocline

The DuQuoin Monocline is located in southwestern Illinois (Figure 2.5-9) where it forms the western boundary of the Fairfield Basin. The monocline has a north-south strike, and dips to the east. The structure extends from the vicinity of DuQuoin to a point approximately 20 miles north of Centralia, a total distance of about 60 miles (Reference 24). The monocline developed during subsidence of the Illinois Basin. There is no evidence of post-Paleozoic movement.

2.5.1.1.4.1.8 Salem and Louden Anticlines

The Salem and Louden Anticlines (Figure 2.5-9) are north-south trending structural highs in the Fairfield Basin. The Salem Anticline extends from central Jefferson County to central Marion County in southern Illinois, and is approximately 20 miles in length. The Louden Anticline is located 7 miles northeast of the Salem Anticline. The Louden Anticline extends from the northern county line of Marion County through east-central Fayette County, Illinois, and is approximately 35 miles long. See Illinois Geological Survey Circular 519.

Pennsylvanian units thin over the Salem and Louden Anticlines, indicating that the two anticlines were uplifted during Pennsylvanian time and later (Reference 22).

2.5.1.1.4.1.9 Clay City Anticline

The Clay City Anticline is a prominent structure in the Fairfield Basin. It trends north-south and has an axial trace approximately 57 miles long. The anticline is a semicontinuous series of anticlinal uplifts separated by saddles (Reference 25). DuBois and Siever (Reference 25) 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 occurred during Pennsylvanian and/or, post-Pennsylvanian time.

2.5.1.1.4.1.10 Dupo-Waterloo Anticline

The Dupo-Waterloo Anticline trends approximately N 20° W from Monroe County, Illinois, through St. Louis, Missouri (Figure 2.5-9). The northern end of the anticline terminates against the Cap au Gres Faulted Flexure (Reference 22). Total structural relief of the anticline is at least 500 feet near Waterloo, Illinois. The western flank of the anticline dips much more steeply, 30° or more, than the eastern flank, which dips only 2° or 3o° (Reference 22). Major movements along the anticline probably occurred from Late Mississippian to pre-Pennsylvanian time, with renewed uplift in post-Pennsylvanian and pre-Pleistocene time (Reference 22).

2.5.1.1.4.1.11 Pittsfield-Hadley Anticline

The Pittsfield-Hadley Anticline trends northwest-southeast and crosses Lewis County, Missouri, and Adams and Pike Counties, Illinois (Figure 2.5-9). Pennsylvanian strata on the flanks of the anticline dip less steeply than those of the underlying Mississippian, suggesting an episode of uplift in the interval from post-Mississippian to pre-Pennsylvanian time, but the major period of uplift is considered to have been post-Pennsylvanian (Reference 22).

2.5.1.1.4.1.12 Cap au Gres Faulted Flexure

The Cap au Gres Faulted Flexure, located some 117 miles southwest of the site, is a structure that 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 of "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 Flexure was post-St. Louis (Mississippian) and pre-Pottsville

(Pennsylvanian) (Reference 26). Later periods of movement may have occurred; however, there is no evidence of deformation of nearby Pleistocene deposits.

2.5.1.1.4.1.13 Sangamon Arch

The Sangamon Arch is located in central and western Illinois (Figure 2.5-9). The crest of the arch trends northeast-southwest across the broad shelf area west of the Illinois Basin and toward the northern center of the Illinois Basin. Buschbach (Reference 22) defines the arch by the zero isopach of the Cedar Valley Limestone (Middle Devonian).

The Sangamon Arch was formed by uplift during the Devonian and Early Mississippian. The arch is a relict structure that has been masked by post-Mississippian movement (Reference 22).

A recent study by Calvert (Reference 27) has questioned the existence of the Sangamon Arch, however the structure is still recognized by the Illinois State Geological Survey.

2.5.1.1.4.1.14 Structures Associated with the Plum River Fault Zone

Four minor structural features, all associated with the Plum River Fault Zone in Illinois, are located approximately 150 miles northwest of the site (Figure 2.5-11). There are similar type structures in adjacent Iowa. Successively from west to east, the Illinois structures are the Uptons Cave Syncline, the Forreston and Brookville Domes, and the Leaf River Anticline. 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, and their conterparts in Iowa, are considered to be associated with the development of the Plum River Fault Zone (References 28 and 29). The Plum River Fault Zone formed in the interval from post-Silurian to pre-Pleistocene time. Based on regional geologic history, the Plum River Fault Zone probably developed at the same time that major movements were occurring on other structures in the region, which was near the beginning of Pennsylvanian time and again in post-Pennsylvanian time (Reference 28). In Illinois, Illinoian Strata of the Plum River Fault Zone shows no evidence of tectonic disturbance (Reference 30); in Iowa there is no known evidence of displacement of Pleistocene deposits (See Iowa Geological Survey letter in Attachment D2.5).

2.5.1.1.4.1.15 Moorman Syncline

The Moorman Syncline is located in southeastern Illinois and northwestern Kentucky, approximately 170 miles south-southeast of the site. It trends roughly east-west, parallel to and south of the Rough Creek Fault Zone. The Moorman Syncline is smaller and narrower than the Fairfield Basin, but somewhat deeper. The major movement on the Moorman Syncline took place in post-Pennsylvanian time (Reference 20). The Moorman Syncline is also known as the Rough Creek Graben.

2.5.1.1.4.1.16 Folds in Wisconsin

Within the regional area are two major folds in southwestern Wisconsin (Figure 2.5-9). The Meekers Grove Anticline trends east-west from Dubuque County, Iowa to Green County, Wisconsin. Its amplitude ranges from 100 to 200 feet. It is approximately 80+ miles long and varies in width from 3 to 5 miles. The north limb of the anticline dips much more steeply than the south limb.

The Mineral Point Anticline is a complexly curved fold in southwestern Wisconsin. It is at least 70 miles long, ranges in width from 5 to 8 miles, and has an amplitude of between 100 and 170 feet. The Mineral Point Anticline is asymmetric with a more steeply dipping north limb. Both the Meekers Grove and Mineral Point Anticlines were probably formed in Late Paleozoic time (Reference 31).

2.5.1.1.4.1.17 Folds in Iowa

Harris and Parker (Reference 32) have delineated five anticlines in southeastern Iowa by virtue of borehole data and structure contours on the Burlington Limestone (Mississippian). The five structures which generally parallel one another are the Bentonsport, Skunk River, Burlington, Sperry, and Oquawka Anticlines. Axial trends of these folds vary from N 55° W to N 65° W. Since 1964 the Iowa Geological Survey has reinterpreted the folded area (Attachment D2.5, Paragraph 3 of letter dated November 6, 1978) and concluded that the structures are not as continuous as shown. There is evidence for a series of northeast-trending folds cutting across the southeast-trending folds. The age of folding is difficult to establish with precision but it is thought to be Mississippian (Reference 32).

2.5.1.1.4.1.18 Minor Folds In Missouri

Several minor folded structures are present in the southeastern Missouri portion of the regional area. These structures have formed at intermittent times during the Paleozoic. Reference 17 and the Clinton PSAR give more precise estimates.

The Troy-Brussels Syncline has an east-west strike (Reference 17) and is located approximately 125 miles southwest of the site (Figure 2.5-9). It extends from near Troy, Missouri to west of Brussels, Illinois. Age of movement was Late Mississippian-Early Pennsylvanian.

The Cuivre Anticline is located southwest of the Troy-Brussels Syncline (Figure 2.5-9). It has an axis which strikes N 80° W, and it plunges to the southeast (Reference 17). It is separated from the Lincoln Anticline by the Troy-Brussels Syncline. Movement was post-Mississipian.

The Eureka-House Springs Anticline is located approximately 150 miles southwest of the site (Figure 2.5-9). The axis of the Eureka-House Springs Anticline trends approximately southeast-northwest, and appears to plunge both to the southeast, in Jefferson County, Missouri, and to the northwest, in St. Louis County, Missouri (Reference 17). Movement was post-Mississippian.

The Farmington Anticline is located approximately 175 miles southwest of the site in Ste. Genevieve and St. Francois Counties, Missouri. The axis of the anticline trends N 30° W, and the anticline extends for a distance of 15 to 20 miles. The northeast limb of the anticline is steeper with dips up to 4°, while the dip on the southwest limb is 1°. The anticline is terminated by faults at both ends. It has been concluded that the Farmington Anticline is no older than Devonian (Reference 17).

The Plattin Creek Anticline is located approximately 160 miles southwest of the site. It trends approximately N 15° E, and is 18 miles long (Reference 17). Movement of the structure is thought to be late Mississippian-Early Pennsylvanian.

The Crystal City Anticline is located approximately 155 miles southwest of the site. The south limb dips at 5° while the north limb has a more gentle 2° dip (Reference 17). Movement is post-Mississippian.

The Krugers Ford Anticline is located approximately 175 miles southwest of the site in Osage and Gasconade Counties, Missouri (Figure 2.5-9). The anticline strikes northeast-southwest with the steep flank on the southeast (Reference 17). Structural movement was post-Ordovician.

The Pershing-Bay-Gerald Anticline is located approximately 180 miles southwest of the site, and trends approximately northwest-southeast (Figure 2.5-9). It passes through the villages of Pershing and Bay, and near Gerald, Missouri to the faulted area near Anaconda (Reference 17). Structural activity was in post-Mississippian and early Pennsylvanian time.

The Mineola Structure is located approximately 165 miles southwest of the site in Montgomery County, Missouri (Figure 2.5-9). The Mineola Structure is an asymmetric dome, trending northwest-southeast, with steeper dips on the southwest side (Reference 17). The structure was active in Pennsylvanian and post-Pennsylvanian time.

The Auxvasse Creek Anticline is located approximately 175 miles southwest of the site (Figure 2.5-9). The Auxvasse Creek Anticline strikes northwest, and is asymmetric with a steep southwest limb (Reference 17). The anticline developed in post-Pennsylvanian time.

The Brown's Station Anticline is located approximately 200 miles southwest of the site (Figure 2.5-9). It trends northwest-southeast. The southwest limb of the anticline is steep and may be faulted (Reference 17). The structure developed in late Mississippian or Pennsylvanian time.

The Davis Creek Anticline is located in Audrain County, Missouri, approximately 180 miles from the site (Figure 2.5-9). The structure trends approximately northwest-southeast and was mapped using subsurface data (Reference 17). The structure was active in post-Mississippian time.

The Mexico Anticline is located in Audrain County, Missouri, approximately 175 miles from the site. This anticline strikes northeast-southwest approximately normal to the general northwest-southeast trending structures of the area (Figure 2.5-9). The anticline is late Pennsylvanian or Pennsylvanian in age.

The College Mound-Bucklin Anticline is located 200 miles west-southwest of the site (Figure 2.5-9). The anticline strikes northwest-southeast and has a gentle plunge to the northwest (Reference 17). It follows the pattern of other such flexures in northern Missouri in having a gentle northeast limb and a slightly steeper southwest limb. Structural development occurred in late Pennsylvanian or post-Pennsylvanian time.

2.5.1.1.4.2 Faulting

Major faults within a 200-mile radius of the site are discussed in the following subsections. Figures 2.5-8 and 2.5-10 show the location and extent of these faults. Table 2.5-2 summarizes the type and displacement of each fault, and gives the age of the last movement.

2.5.1.1.4.2.1 Sandwich Fault Zone

The Sandwich Fault Zone is located in northern Illinois, and strikes west-northwest from western Will County, Illinois to Ogle County, Illinois (References 22 and 30). The closest approach to the site is approximately 90 miles north-northeast (Figure 2.5-10). The northeast side of the fault zone is down thrown, to a maximum displacement of 800 feet (Reference 30). The fault zone forms the northern boundary of the Ashton Arch. 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 during post-Mississippian to pre-Pennsylvanian time (References 22 and 30). There is no relationship between historical earthquake epicenters and the Sandwich Fault Zone (Reference 30).

2.5.1.1.4.2.2 Plum River Fault Zone

The Plum River Fault Zone, (formerly called the Savanna Fault and the Savanna Anticline) is a generally east-west trending zone of high angle faults extending from near Leaf River (Ogle County), Illinois, west-southwest into Southern Linn County, Iowa (References 28 and 29, Figures 2.5-10 and 2.5-11). The width of the fault zone is less than one-half mile, and vertical displacement along the fault zone is 100 to 400 feet, north side down (Reference 28). The age of movement in Illinois has been limited to post-middle Silurian to pre-middle Illinoian (Pleistocene) (Reference 28). However, there are no deposits representing the interval from middle Silurian to middle Illinoian (Pleistocene) time, and thus, more precise dating of the faulting is not possible. It seems likely that the Plum River Fault Zone formed at the same time as the La Salle Anticlinal Belt and Mississippi River Arch, near the beginning of Pennsylvanian time and again in post-Pennsylvanian time (Reference 28). In Iowa there is evidence of Paleozoic movement of the Plum River Fault Zone (Reference 29) but no known evidence of displacement of Pleistocene strata (Attachment D2.5, Paragraph 1 of letter dated November 6, 1978).

Minor folded structures are associated with the Plum River Fault Zone. In Illinois, named folds are the Forreston Dome, the Brookville Dome, the Leaf River Anticline, and Uptons Cave Syncline. These structures are discussed in Subsection 2.5.1.1.4.1.14.

2.5.1.1.4.2.3 Cap au Gres Faulted Flexure

The Cap au Gres Faulted Flexure is discussed in Subsection 2.5.1.1.4.1.12.

2.5.1.1.4.2.4 Centralia Fault

The Centralia Fault is located in Marion and Jefferson Counties, Illinois. It is a zone of several parallel faults, striking north-south, which are parallel to, and 1 mile east of the DuQuoin Monocline (Reference 22). The closest approach of the fault is approximately 110 miles south of the site (Figure 2.5-10). Downthrow on the fault is 160 to 200 feet on the west (Reference 33). Although faults have been observed in several coal mines in the Centralia area, they have no surface expression (References 22 and 33). The faulting occurred in the interval from post-Pennsylvanian to pre-Pleistocene time (Reference 22).

2.5.1.1.4.2.5 Ste. Genevieve Fault Zone

The Ste. Genevieve Fault Zone trends approximately northwest-southeast from Perry County, Missouri to Union County, Illinois, and is located approximately 160 miles southwest of the site (Figure 2.5-10). The faults are high angle and form numerous horsts and grabens. The net displacement is down to the north and east with the maximum displacement greater than 1000 feet, and possibly up to 2000 feet. The Ste. Genevieve Fault Zone forms a sharp boundary, a few miles wide, between the Illinois Basin and the Ozark Uplift. Movement on the Ste. Genevieve Fault Zone may have started as early as the Devonian, and movement is known to have occurred both in the post-Mississippian, pre-Pennsylvanian interval and in post-Pennsylvanian time (Reference 22).

2.5.1.1.4.2.6 Rough Creek Fault Zone

Buschbach (Reference 22) states that the Rough Creek Lineament is a series of faults and fault zones extending generally east-west through western Kentucky and southern Illinois. In Kentucky, it includes the Rough Creek Fault Zone. In Illinois, it includes the east-west portion of the Shawneetown Fault Zone to the east, and the Cottage Grove Fault System to the west. Heyl (Reference 34) suggests that strike-slip faulting or wrench faulting is a major component in the Rough Creek Lineament, however, this has been disputed. Heyl tentatively includes it in a line or zone of faults, monoclines, and igneous intrusions. The line extends east-west for 800 miles along the 38th parallel from West Virginia to at least as far west as the Ozark Uplift.

In Illinois, the lineament includes numerous high angle reverse faults with the south side upthrown. They appear to be the result of compressional forces from the south, and they display evidence of some horizontal movements. The eastern part of the lineament, the Shawneetown Fault Zone, is dominated by high angle thrust faulting. Displacement is locally as great as 3400 feet and may be considerably more. The Shawneetown Fault Zone extends westward along the prominent hills in southern Gallatin County, curves southward around Cave Hill in Saline County, leaves the Rough Creek Lineament and joins the southwest trending Herod Fault to form the Lusk Creek Fault Zone. The western portion of the lineament, the Cottage Grove Fault System, appears to have formed at roughly the same time as the Shawneetown, but displacements are much diminished, with maximum displacements of about 250 feet. From all available evidence it appears that the age of faulting along the Rough Creek Lineament is chiefly post-Pennsylvanian, pre-Late Cretaceous, although some workers have suggested the possibility of later movements because of recent seismic activity in the general area.

2.5.1.1.4.2.7 Wabash Valley Fault System

The Wabash Valley Fault System is a series of generally parallel faults, 125 miles from the site at its closest approach, that is terminated at the Rough Creek Fault Zone (Reference 35). It extends north-northeastward from the Rough Creek Fault Zone generally parallel to the Wabash River in southeastern Illinois and southwestern Indiana (Figure 2.5-10). The faults are high angle, normal faults (Reference 22), some of which border horsts and grabens (Reference 36). Maximum displacement known on the faults is a little over 400 feet, but displacements from a few to 200 feet are more common. Displacements of Mississippi and Pennsylvanian age strata along the faults is the same, indicating that the faults are post-Pennsylvanian in age. No displacement has been recognized in Pleistocene deposits, thus the faulting was concluded in pre-Pleistocene time (Reference 22).

2.5.1.1.4.2.8 Northeast Trending Faults South of the Rough Creek Fault Zone

South of the Rough Creek Fault Zone, in southeastern Illinois and western Kentucky, is an intensely faulted area (Figure 2.5-10). Faults in this area trend dominantly northeast-southwest and east-west. Both normal and reverse faults are found in this area (Reference 37), but most of the faults are normal faults (Reference 38). Some graben structures are known in the area (Reference 37). Displacements on faults in the area are variable (Reference 38), but displacements up to 2000 feet have been reported (Reference 37). Faults south of the Rough Creek Fault Zone are generally post-Pennsylvanian in age and terminate beneath the Cretaceous cover of the Mississippi Embayment, but there is some evidence that faulting on a small scale continued into the Tertiary (Reference 14).

2.5.1.1.4.2.9 Mt. Carmel Fault

The Mt. Carmel Fault trends approximately N 25° W across south central Indiana from Washington County, north to Monroe County, approximately 130 miles southeast of the site (Figure 2.5-10). It is a normal fault, downthrown to the west, with a displacement which may be in excess of 200 feet. Movement along the fault may have begun in late Mississippian time and probably was concluded by Early Pennsylvanian time (Reference 39).

2.5.1.1.4.2.10 Fortville Fault

The Fortville Fault trends northeast-southwest through central Indiana, approximately 145 miles east of the site (Figure 2.5-10). The Fortville Fault is a normal fault and is about 54 miles long. On the basis of structure contours on the top of the Ordovician Trenton Limestone (Reference 40), the fault has a vertical displacement of about 60 feet, downthrown to the southeast (Reference 41). The fault displaces Devonian age strata, but does not displace overlying Pleistocene age deposits (Reference 42).

2.5.1.1.4.2.11 Royal Center Fault

The Royal Center Fault trends northeast-southwest for 47 miles in north-central Indiana. It is approximately 125 miles from the site at its closest approach (Figure 2.5-10). On the basis of structure contours on the top of the Ordovician Trenton Limestone (Reference 40), the Royal Center Fault is a normal fault, downthrown to the southeast, with a vertical displacement of about 100 feet (Reference 41). The fault displaces Devonian age strata, but does not displace overlying Pleistocene age deposits (Reference 42).

2.5.1.1.4.2.12 Localized Faults in Wisconsin

Locally restricted faults are known in the regional area from two locales in southern Wisconsin: southern Dane County, Wisconsin, and at Waukesha, Wisconsin. These faults are not shown in Figure 2.5-10.

The locally restricted faults reported in southern Dane County, Wisconsin trend northeast. Displacements on these faults in the southern Madison, Wisconsin area are approximately 150 feet, and in the Mt. Vernon, Wisconsin area, about 60 feet (Reference 43). Displacements in the general southern Dane County area may be as great as 300 feet with the northwest side down (Reference 44). The age of the faulting in southern Dane County is post-Ordovician pre-Pleistocene (References 43, 44). These faults in southern Dane County do not correlate with the regionally continuous Madison fault reported by Thwaites (Subsection 2.5.1.1.4.2.15), the existence of which is now considered doubtful by present workers (Reference 43).

Faulting is known at an exposure in Waukesha, Wisconsin, where the southeast side of the fault has been downthrown 27 feet. Movement on the fault is considered to have occurred in post-Silurian pre-Pleistocene time. Subsurface data does not substantiate extending this fault past the Waukesha town limits. This fault was originally the basis of an inferred fault in the Precambrian basement (the "Waukesha" Fault, described in Subsection 2.5.1.1.4.2.15), but recent studies reject the interpretation of a major fault extending from the Waukesha area (Reference 43).

2.5.1.1.4.2.13 Cryptovolcanic or Astrobleme Structures

Cryptovolcanic or astrobleme structures, shown in Figures 2.5-8 and 2.5-10, include the Glasford, Kentland, and Des Plaines Disturbances. These are complex, local areas of intense shattering, thought to be due to meteorite impact.

The Glasford Disturbance is approximately 60 miles northwest of the site. The Glasford Disturbance has been outlined by gravity surveying and structural drilling; the disturbance consists of a dome with a normal sequence of Upper and Middle Paleozoic strata underlain by a series of jumbled blocks in a breccia matrix of Cambrian formations uplifted about 1000 feet. Middle and Lower Ordovician strata are not recognizable in the disturbed zone. The chaotic condition of the pre-Upper Ordovician rocks suggests that this disturbance was caused by a violent explosion which probably took place in early-Late Ordovician time (Reference 45). This feature is 2-1/2 miles in diameter and is very local in nature. The Glasford Disturbance is the closest structure of this type to the site.

The Des Plaines Disturbance is located in northeastern Illinois, approximately 140 miles from the site (Figure 2.5-10). The disturbance is roughly circular, about 5-1/2 miles in diameter, and contains many small blocks separated by normal and reverse faults. A graben, in which Mississippian and Pennsylvanian age rocks are preserved, partly surrounds a central uplifted core. The Des Plaines Disturbance was formed sometime in the interval from post-Pennsylvanian to pre-Wisconsinan (Pleistocene) time (Reference 46).

The Kentland Disturbance is located in western Indiana, approximately 90 miles from the site. As in other cryptovolcanic structures, the Kentland Disturbance is characterized by a nearly circular outline, a central uplift, and a marginal, ring-shaped depression with irregular and local faulting (Reference 15). The Kentland Disturbance is considered to be late Paleozoic or Mesozoic in age.

2.5.1.1.4.2.14 Postulated Faults in Illinois

2.5.1.1.4.2.14.1 Oglesby Fault and Tuscola Fault

From maps of the Trenton (Ordovician) structure in Ohio, Indiana, and northern Illinois, Green (Reference 47) postulated two faults in Illinois, the Oglesby Fault and the Tuscola Fault (Figure 2.5-10). These two faults approximately coincide with the western flank of the La Salle Anticlinal Belt, and the western sides of these faults were inferred to be downthrown. The Illinois State Geological Survey has never accepted the existence of the Oglesby and Tuscola Faults, and has found no evidence to support such an interpretation. The Illinois State Geological Survey contends that the differences in elevations of the top of the Trenton are not due to faulting, as suggested by Green, but are simply due to dipping beds with dips of a few to 10° (Reference 48).

2.5.1.1.4.2.14.2 Chicago Area Basement Fault

On the basis of gravity and seismic geophysical evidence, McGinnis postulated a basement fault zone in the metropolitan Chicago area north of and about paralell to the Sandwich Fault Zone (Figure 2.5-10). It was inferred that the southwest side of this fault zone was downthrown up to 900 feet. The presence of this basement fault has not been verified. McGinnis (Reference 49) has postulated that movement on this basement fault was completed by Middle Ordovician time.

2.5.1.1.4.2.14.3 Chicago Area Minor Faults

As a result of a recent seismic survey in the metropolitan Chicago area, 25 faults were inferred, with displacements up to 55 feet. None of these faults involves wide shear zones or detectable scarps on the rock surface (Reference 50).

Other 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 show less than 1 foot of movement (Reference 50).

2.5.1.1.4.2.15 Postulated Faults in Wisconsin

Thwaites' map of the buried Precambrian surface in Wisconsin (Reference 51) postulates the existence of several faulted areas in southern Wisconsin. Ostrom (Reference 43) stated that Thwaites' map is diagrammatic and does not represent a detailed study of each fault. Three of the postulated faults, named by Dames & Moore as the Janesville, Madison, and Waukesha Faults, have a noticeable difference in the elevation of the Precambrian basement, which was interpreted by Thwaites to be the result of faulting. It is now believed (Reference 43) that this difference in the elevation of the Precambrian basement is not due to faulting, but due to topographic relief on an erosional Precambrian basement surface.

2.5.1.1.5 Gravity and Magnetic Anomalies

Gravity anomalies are caused by lateral rock density changes which may result from:

a. structure, unconformities, and lithologic changes in the sedimentary rocks;

b. igneous intrusives;

c. relief on the crystalline basement surface; and

d. lithologic changes in the crystalline portion of the earth's crust and upper mantle.

Figure 2.5-12 represents the Bouguer gravity anomaly map of the region surrounding the site. The anomaly pattern north of 38.5-39° is typically midcontinental in style (Reference 52). The site lies on the flank of a moderate gravity low associated with the La Salle Anticlinal Belt where dense Precambrian crystalline rocks lie at relatively shallow depths beneath the surface (Reference 2).

Magnetic anomalies are largely caused by lateral changes in the concentration of the mineral magnetite. Therefore, in the site region, magnetic anomalies result from:

a. igneous intrusives;

b. relief on the crystalline basement surface; and

c. lithologic changes in the crystalline portion of the earth's crust above the Curie point isotherm.

Detailed aeromagnetic surveys have been carried out in Illinois only south of 39° N (References 54 and 55). An early ground magnetic survey (Reference 56) is shown in Figure 2.5-13. The scale of the gravity anomaly map does not permit anything other than a general correlation between gravity anomalies and major regional structures.

2.5.1.1.6 Regional Groundwater

Groundwater conditions in the vicinity of the site are discussed in Subsection 2.4.13.

2.5.1.1.7 Man's Activities

A discussion of man's activities in the site area is included in Subsection 2.5.1.2.7.

2.5.1.2 Site Geology

2.5.1.2.1 Site Physiography

The site lies within the Bloomington Ridged Plain physiographic subsection of the Till Plains Section (Figure 2.5-1). The main plant is located in an area of uplands, consisting of Wisconsinan-age ground moraine, that have been dissected by the Salt Creek and the North Fork of the Salt Creek (Figures 2.5-14 and 2.5-15). The ultimate heat sink and associated structures are located along the North Fork of the Salt Creek (Figures 2.5-14 and 2.5-16), and the main dam is located below the confluence of the Salt Creek and the North Fork of the Salt Creek (Figure 2.5-14).

The uplands consist of gently rolling ground moraine, located just east of the Shelbyville end moraine, with local relief of about 10 feet, except near the drainageways. Average elevation of the uplands is approximately 740 feet MSL.

Two perennial streams, Salt Creek and North Fork of the Salt Creek are present in the site area. The two streams join in the southern portion of the site area (Figure 2.5-14). The two streams flow generally to the southwest with gradients of 2 to 3 feet per mile in the site area. They have eroded through the upland deposits of the Wisconsinan-age Wedron Formation and Robein Silt, the Illinoian-age weathered Glasford Formation, and into the upper part of the Illinoian-age unaltered Glasford Formation. The elevation of the floodplains of the two streams in the site area is at approximately 660 feet MSL. Maximum relief in the site area is on the order of 80 feet.

2.5.1.2.2 Site Stratigraphy

The general stratigraphy of the Clinton Power Station (CPS) site is shown in Figure 2.5-18, Site Stratigraphic Column and a detailed description of stratigraphic units exposed in plant excavation is presented in Attachment C2.5. Boring logs are presented in Figures 2.5-19 through 2.5-270. Topography, plant structures, plot plans, and location of geologic sections across the site are given in Figures 2.5-271, 2.5-272, 2.5-273, and 2.5-16. The strata underlying the site consist of overburden deposits, about 225 to 360 feet in thickness in the upland areas, resting on Pennsylvanian-age bedrock.

The overburden materials, in order of increasing age, consist of stream alluvium, windblown loess, and glacial drift. Colluvium and glacial outwash are also present. The stratigraphic nomenclature of the overburden deposits used for the excavation mapping reports (Attachment C2.5) and the FSAR is different from that used for the PSAR. This change was brought about in order that the nomenclature of the site glacial drift would be consistent with that used by the Illinois State Geological Survey. Figure 2.5-274 is a chart showing the correlation between stratigraphic terms used in the PSAR and those used in the FSAR and excavation mapping reports. The terminology changes do not indicate any difference between the lithologic units encountered in the borings and those encountered in the excavations.

Overburden materials and bedrock deposits are discussed in greater detail in the following subsections.

2.5.1.2.2.1 Overburden Materials

Overburden materials occurring in the site vicinity, in order of increasing age, consist of stream alluvium, windblown loess, and glacial drift. Colluviam and glacial outwash are also present. Figures 2.5-275 through 2.5-280 show cross sections which portray the relationships of the various overburden materials across the site area, as determined from the boring logs. Attachment C2.5 describes the overburden materials exposed in excavations for the Clinton Power Station.

Addressing valley deposits the stream alluvium and recent channel deposits, known as the Cahokia Alluvium, are composed of poorly sorted silt, clay, and silty sand, with sand and gravel lenses. Beneath the alluvium are glacial outwash deposits of the Henry Formation consisting of yellow-brown fine to coarse sand and gravel, pockets of silty-clayey material, and a basal lag gravel. Not differentiated in field mapping, these deposits range up to 35 feet in thickness, collectively, in the site borings and excavations, and are restricted to the valleys of Salt Creek and North Fork of Salt Creek. Also in the valleys, at the base of the valley walls, are local deposits of Peyton Colluvium directly overlying the Cahokia Alluvium and, in-places, Illinoian Till of the Glasford Formation. The colluvium is composed of brown clayey silts with minor amounts of gravel. In the excavations for structures for the Clinton Power Station, (in the valleys), the alluvium-outwash colluvium was underlain by unaltered Illinoian-age Glacial Till of the Glasford Formation.

The Cahokia Alluvium, Henry Formation, and Peyton Colluvium were referred to as the Salt Creek Alluvium, Flood Plain Alluvium, or Recent Channel Deposits in the PSAR text and as the Salt Creek Alluvium on the boring logs (Figure 2.5-274). The Cahokia Alluvium is Holocene and possibly, in part, Valderan/Twocreekan in age; the Henry Formation is Woodfordian (probably early) in age.

In the uplands between drainages, loess and glacial drift comprise the surficial deposits. Total drift thickness in the uplands varies from 210 to 310 feet in borings, this difference being largely due to the relief on the bedrock surface beneath the glacial drift (Figures 2.5-7 and 2.5-281).

The loess, known as the Richland Loess, consists, generally, of a brown clayey silt with a trace of sand, and is present in the uplands at the site to thicknesses of 5 to 10 feet. A modern soil profile has been developed in the Richland Loess. Both the Richland Loess and the Wedron Formation which it overlies were deposited during the Wisconsinan Stage of glaciation. The Richland Loess was referred to as "Loess" in the PSAR text and on the boring logs. The Richland Loess may be Holocene in age, in part.

The deposits of glacial drift in the site area form a complex sequence of materials (Figure 2.5-18). The uppermost deposits confined to the upland consist of Wisconsinan-age glacial till of the Wedron Formation. The Wedron Formation was referred to as Wisconsinan Glacial Till in the PSAR and boring logs (Figure 2.5-274). The Wedron Formation is from 20 to 55 feet thick in the site area where it has not been partially removed by erosion. It is composed of stiff to very stiff clayey sandy silt till, that is brown in cloration in the upper oxidized zone but grades to gray in the unoxidized zone. Discontinuous lenses of stratified sand, silt or gravel are randomly interbedded within the till of the Wedron Formation at the CPS site.

Underlying the till of the Wedron Formation is the Robein Silt. The Robein Silt, also restricted to the uplands, was deposited during the Farmdalian Substage of the Wisconsinan Stage. It is a dark colored silt, rich in organic material. It is present over much of the site area, and may be up to 2 feet thick, although locally it may be absent due to erosion. The Farmdale Soil (Reference 5) is developed in the Robein Silt. The Robein Silt was previously included with the "interglacial zone" in the boring logs and the "Sangamonian interglacial soil zone" or "Sangamon soil interval" in the PSAR text (Figure 2.5-274).

Underlying the Robein Silt, and also present under the valleys, are deposits of Illinoian-age collectively referred to as the Glasford Formation. The upper part of the Glasford Formation was weathered during the late Illinoian Sangamonian and possibly Altonian stages, and these weathered deposits are referred to as the weathered Glasford Formation in the USAR and in the excavation mapping reports (Attachment C2.5) and as Interglacial Zone, Sangamon Interglacial Zone, or Sangamon Soil Interval in the PSAR. Preserved mostly in the uplands, the weathered Glasford Formation is leached, characteristically black, dark brown, green or bluish-green and is 10 to 15 feet thick in the site area. The weathered materials are dominantly glacial till, consisting of silty clay and clayey silt but locally they may be discontinuous lenses of silts, sands, or sandy silts interbedded within the glacial till of the Glasford Formation.

The boundary between the weathered Glasford Formation and the unaltered Glasford Formation is marked by the occurrence of calcareous glacial till of the Glasford Formation. The unaltered Glasford Formation at the site ranges in thickness from 90 to more than 140 feet. It is dominantly a hard, gray-brown sandy silt till. Discontinuous layers of stratified sand, gravel, or silt, up to 2 to 3 feet in thickness may be interbedded within the till in the uppermost part of the unaltered Glasford Formation at the site (Attachment C2.5). The lower part of the unaltered Glasford Formation exposed in the excavations appears to have virtually no interbedded stratified deposits. Excavations for plant structures extended down into only the unaltered Glasford Formation, and data on the underlying deposits are from borehole samples. The unaltered Glasford Formation was referred to as "Illinoian Till" or "Illinoian Glacial Till" in the PSAR text and on the boring logs (Figure 2.5-274).

Beneath the Glasford Formation is a complex assemblage of glacial materials consisting of gray to brown clay till (which is occasionally sandy), reworked till and outwash, and glacio-lacustrine gray silt. Correlation of these formations throughout the site area is difficult and uncertain. The sequence is probably pre-Illinoian in age and varies in thickness from 10 to 105 feet. These materials have been tentatively assigned to the Banner Formation which was deposited during the Kansan Stage (Figure 2.5-18). (Existing classifications of pre-Illinoian Plastocene glacial deposits have been questioned by some as noted in Reference 4).

In some areas of the site, as beneath the main power block, the complex of probable pre-Illinoian till, outwash, and glaciolacustrine deposits lies in direct contact with bedrock. Generally, however, it is underlain by a clean sand and gravel deposit of highly variable thickness which is identified as Kansan Stage glacial outwash (Mahomet Sand Member of the Banner Formation). This interval shows pronounced thickening where the bedrock surface slopes to lower elevations and is a glaciofluvial filling in the bedrock valleys. Its thickness ranges from zero on the highest bedrock surfaces to 140 feet at the lowest bedrock elevations.

2.5.1.2.2.2 Bedrock Formations

Bedrock was penetrated by 19 of the borings. Twelve borings were located in the area of the power block (Figure 2.5-271), two at or near the dam site (Figure 2.5-272), and three in the ultimate heat sink area (Figure 2.5-16). One boring was drilled about 2 miles northeast of the station site for the purpose of identifying the regional trend of drift thicknesses and bedrock lithologies.

The site is underlain by bedrock of Pennsylvanian age. The bedrock surface at the site is an erosional surface that varies in elevation from 360 to 510 feet MSL. Figure 2.5-281 shows the general configuration of the bedrock surface in the Dewitt-McLean County area as determined from boring logs and geophysical data. Data from borings for the CPS site have necessitated some modifications to the location of the bedrock valleys that are shown in the site area in Figure 2.5-281. The bedrock surface in the site area, as determined from these borings, is shown in Figures 2.5-17 and 2.5-282.

The Pennsylvanian bedrock beneath the CPS site is characterized by sharp vertical changes in rock type and by laterial persistence of units such as limestones or coals, where they have not been removed by erosion. Regional marker beds which were encountered during the site investigation are the Shoal Creek Limestone Member, the No. 8 Coal Member, and the No. 7 Coal Member; these units were identified by the Illinois State Geological Survey.

The uppermost Pennsylvanian strata in the site area belong to the Bond Formation of the McLeansboro Group. The Shoal Creek Limestone Member is a marker bed at the base of the Bond Formation. The Shoal Creek Limestone Member is found beneath the power block and ultimate heat sink in areas where the Pennsylvanian bedrock has not been eroded below an elevation of 495 feet MSL. The Shoal Creek Limestone Member is a fine to coarse crystalline limestone with irregular shale partings; in the upper portion of the unit, there are numerous open and clay-filled weathered bedding planes.

Underlying the Bond Formation is the Modesto Formation which is also part of the McLeansboro Group. Three distinctive units of the Modesto Formation were found in the site area. The upper part of the Formation contains an unnamed limestone unit that is continuous across the CPS site in areas where the bedrock has not been eroded below an elevation of 472 feet MSL. This limestone is an argillaceous, fine crystalline limestone, which is variable in thickness, and contains interbedded shale. The No. 8 Coal Member of the Modesto Formation is 1 foot thick at the CPS site, and was encountered at elevation 431 feet MSL in Boring P-38. Below elevation 424 feet MSL at the site, the Modesto Formation is predominantly siltstone and shale. A limestone bed was encountered in Boring D-11, however, at an elevation of 360 feet MSL.

Underlying the McLeansboro Group is the Kewanee Group, which is also of Pennsylvanian age. The uppermost formation in the Kewanee Group is the Carbondale Formation whose top is marked by the No. 7 Coal Member. This unit is 2.5 to 3 feet thick in the site area and was encountered and correlated in the three borings across the site. A chemical analysis was performed on samples from Boring P-38 of the No. 7 and No. 8 Coal Members by the Illinois State Geological Survey, and the results are presented in Table 2.5-3.

Information on Pennsylvanian strata below the No. 7 Coal Member and on older pre-Pennsylvanian rocks was derived from a deep boring (1,621 feet) drilled approximately one mile west of the site (NW part of Section 27, T.20N., R.3E). About 440 feet of Pennsylvanian-aged strata, consisting principally of cyclothems, lie below the No. 7 Coal Member. Below the Pennsylvanian, the boring penetrated some 560 feet of Mississippian shale and limestone, which were in turn underlain by 180 feet of Devonian limestone and shale. The drill hole terminated in Silurian dolomite. Regional correlation between this boring, other deep borings in the vicinity, and borings at the site is shown in Figure 2.5-425. See Subsection 2.5.1.2.5 for information on older strata beneath the site.

2.5.1.2.3 Site Structural Geology

The site is located to the southwest of the Downs Anticline, a subsidiary fold to the LaSalle Anticlinal Belt (Figure 2.5-285). The Downs Anticline trends south to south-southeastward from a point approximately 10 miles north of Bloomington, Illinois to a point approximately 4 miles south of DeLand, Illinois. At its closest the anticline is 4 miles northeast of the site (Figure 2.5-285). Several small domes are located along the axis of the Downs Anticline (Reference 57). Three of these domes lie between 5 and 10 miles from the site: the Wapella East Dome to the northwest, the Parnell dome to the northeast, and the DeLand Dome to the southeast.

The Downs Anticline is an asymmetrical, almost monoclinal fold. Cross sections through the Downs Anticline are shown in Figures 2.5-286 and 2.5-283. Although these figures have considerable vertical exaggeration in order to show the structure, the tops of Trenton (Ordovician) and Hunton (Silurian-Devonian) dip only at about 3° and 2°, respectively, and the dips on the No. 2 Coal Member and the No. 7 Coal Member are even less (about 1/2° to 1°). Rocks below and including the Hunton (Silurian-Devonian) were folded between post-Middle Devonian and pre-Pennsylvanian time, as shown by the accentuated bending of tops of the Trenton (Ordovician) and Hunton (Silurian-Devonian) in Figure 2.5-283. Rocks below and including the coal beds were folded between Pennsylvanian and pre-Pleistocene time. The Pleistocene stratum (Robein Silt) shown in Figure 2.5-283 does not reflect the folding of the coal beds. No tectonic folding or faulting was observed in the Pleistocene deposits exposed in the excavations at the CPS site, including the Robein Silt. The irregularity shown on the Pleistocene stratum shown in Figure 2.5-283 is due to deposition on an uneven erosional surface.

The bedrock surface is an erosional surface, and in the site area there is no general relationship between Paleozoic structures and bedrock topography. However, the Downs Anticline is located over a broad bedrock high, and a small circular bedrock high coincides with the Wapella East Dome. The relation of the other structural domes to bedrock topography is not as obvious.

The DeLand Dome is located in broad bedrock uplands and the Parnell Dome is located on the flank of a bedrock valley. The location of these structures with respect to bedrock topography is shown in Figure 2.5-281. In some cases, structures and bedrock topography coincide; whereas in others, there is no relationship. Structure cannot, therefore, be inferred from bedrock topography.

Borings at the site encountered small elevation differences in certain lithologic units of the bedrock as shown in Figures 2.5-275, 2.5-276, 2.5-279, and 2.5-284. Measured dips on these stratigraphic horizons corresponding to such elevation differences are less than 1° (Figure 2.5-283) and average about 1/2°. In the heat sink area, a dip of 1.9° to the south was determined. A vertical exaggeration of 16 to 1 on the cross sections gives an exaggerated impression as to the magnitude of dip. The bedding slopes away from a high point beneath the site, and this possibly could be indicative of a minor structure. It is more probable, however, that this condition reflects normal irregularities typical of sedimentary contacts, even in tectonically undisturbed regions. Lensing and lithologic interfingering could easily account for such variations without the mechanism of uplift, compressional forces, or faulting. Borings D-31, H-6, and P-38 were drilled into or through the No. 7 Coal Member. The elevation of the coal in each boring correlates reasonably well with the structure contour map on top of the No. 7 Coal Member prepared by Clegg (Reference 57), Figure 2.5-285. The structural contour map on top of the No. 2 Coal Member correlates with structure contour maps on top of the No. 2 Coal Member (Reference 57) and on top of the Middle Devonian strata (Reference 58) except that the lower horizons are more intensely folded.

No faulting has been recognized in association with the foregoing structural features either from aerial photographs, ERTS imagery, geophysical studies, borehole control, or excavation mapping. The glacial materials are devoid of lineaments or off-sets suggestive of faulting. Even if the bedrock unit elevation differences could be attributed to structural deformation, the relatively flat-lying and undeformed Pleistocene drift overlying bedrock demonstrates that the stresses which would have been responsible for the deformation have been inactive since at least pre-Pleistocene time.

The Downs Anticline and its associated axial domes are stable and are of no structural significance at the site.

2.5.1.2.4 Site Surficial Geology

The surficial geology is shown in Figure 2.5-15 and is discussed in Subsection 2.5.1.2.2.1.

2.5.1.2.5 Site Geologic History

The geologic history of the site area is derived partly from exposures of soil units and partly from onsite borings in which samples of soil and bedrock were obtained. The deepest borings within the site area penetrated the uppermost bedrock formations of Pennsylvanian age. Discussions of events which occurred at the site during pre-Pennsylvanian times are based on data derived from adjacent regions.

2.5.1.2.5.1 Precambrian Era (Greater than approximately 600 million Years B.P.)

The Precambrian is the oldest recognized division of geologic time. The history of events which occurred during this long period of time is obscured by deep burial under younger rocks. No borings at or near the site have reached Precambrian rocks. Data from the regional area suggest, however, that the Precambrian rocks in Illinois are igneous rocks, composed of granite, rhyolite, and associated rocks that formed in the interval from 1.1 to 1.4 billion years ago (Reference 2). The Precambrian basement in Illinois underwent a long period of erosion lasting from Late Precambrian time into Cambrian time (Reference 2). Consequently, the Precambrian surface is, in part, an erosional surface which may have several hundred feet of relief (Reference 2). The elevation of the Precambrian basement in the site vicinity is estimated to be approximately -6000 feet MSL (Reference 2) at a depth of approximately 6700 feet.

2.5.1.2.5.2 Paleozoic Era (Approximately 600 to 225 ± 5 million years B.P.

The Clinton Power Station is located in the northern part of the Illinois Basin. This area was subject to intermittent tectonic movements occurring in the Illinois Basin throughout Paleozoic time.

2.5.1.2.5.2.1 Cambrian Period (Approximately 600 to 500 million years B.P.)

No onsite borings have penetrated Cambrian-age deposits. 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 carbonate rocks were deposited. At the close of Cambrian time, the site area was uplifed. This was followed by a brief period of erosion (Reference 2). Approximately 3100 feet of Late Cambrian sediments underlie the site (Reference 2).

2.5.1.2.5.2.2 Ordovician Period (Approximately 500 to 430 ± 10 million years B.P.)

No borings at the site have reached Ordovician age deposits. The Ordovician Period began with a transgression of the sea. General conditions favored the accumulation of calcereous deposits. At the close of Early Ordovician time, the sea again receded and a prolonged period of erosion was initiated. Later, the readvancing sea deposited a considerable quantity of fine to medium sand (the St. Peter Sandstone), followed by a thick sequence of calcareous deposits, and ending with accumulations of silt and clay. Approximately 1000 feet of Ordovician sediments underlie the site (Reference 2).

2.5.1.2.5.2.3 Silurian Period (430 ± 10 to 400 ± 10 million years B.P.)

No borings at the site have reached Silurian-age deposits. However, much of the Illinois Basin, including the site area, was continuously beneath shallow seas during the Silurian Period. Depositon in these shallow seas consisted primarily of carbonates, and reefs developed in some areas. Regional data suggest that carbonate deposition continued from Silurian into Devonian time. Approximately 450 feet of Silurian age sediments underlie the site (Reference 2).

2.5.1.2.5.2.4 Devonian Period (400 ± 10 to 340 ± 10 million years B.P.)

No borings at the site have reached Devonian-age deposits. A major period of uplift and erosion occurred between the end of Lower Devonian sedimentation and the beginning of Middle Devonian sedimentation in the regional area, including the site vicinity. Sedimentation in shallow Middle Devonian seas in the site area probably consisted of limestones and dolomites changing to black and gray shales in Late Devonian time. Total thickness of Devonian strata in the site area is about 200 feet (Reference 2).

2.5.1.2.5.2.5 Mississippian Period (340 ± 10 to 320 ± 10 million years B.P.)

No borings at the site have reached Mississippian age deposits. During Mississippian time, the deposition of fine clastics continued, although conditions gradually changed to alternating deposits of carbonates, silt, and sand. The close of Mississippian time was marked by uplift and erosion. Uplift of the Wapella East Dome portion of the Downs Anticline, near the site area (Subsection 2.5.1.2.3), took place during or prior to Mississippian time, and continued after the deposition of Mississippian age sediments (Reference 57), prior to Pennsylvanian deposition.

2.5.1.2.5.2.6 Pennsylvanian Period (320 ± 10 to 270 ± 5 million years B.P.)

The cyclical units of Pennsylvanian age strata indicate alternating periods of marine, nearshore, deltaic, and continental deposition which took place in much of the regional area throughout Pennsylvanian time. Nearly 600 feet of Pennsylvanian age strata were deposited in the site area (Reference 2).

Uplift on the Wapella East Dome portion of the Downs Anticline near the site occurred during and/or after Pennsylvanian time (Reference 57). A comparison of the relative amount of uplift occurring on the Downs Anticline before Pennsylvanian deposition and during and/or after Pennsylvanian deposition can be made by comparing the profiles of the tops of the Trenton (Ordovician) and Hunton (Silurian-Devonian) with the tops of the No. 2 and No. 7 Coal Members (Pennsylvanian) in Figures 2.5-283 and 2.5-286.

2.5.1.2.5.2.7 Permian Period (270 ± 5 to 225 ± 5 million years B.P.)

There are no deposits of Permian age in the regional area. It has been speculated that some Permian age deposits might have been deposited in the Illinois Basin (References 2 and 14), but if so, they were completely removed by subsequent erosion.

2.5.1.2.5.3 Mesozoic Era (225 ± 5 to 65 ± 2 million years B.P.)

2.5.1.2.5.3.1 Triassic Period (225 ± 5 to 190 ± 5 million years B.P.)

There are no deposits of Triassic age at the site. This was largely a period of erosion throughout the regional area (Reference 2).

2.5.1.2.5.3.2 Jurassic Period (190 ± 5 to 135 ± 5 million years B.P.)

There are no deposits of Jurassic age at the site. This was largely a period of erosion throughout the regional area (Reference 2).

2.5.1.2.5.3.3 Cretaceous Period (135 ± 5 to 65 ± 2 million years B.P.)

There are no Cretaceous age deposits at the site. This was probably a period of erosion in the site area.

2.5.1.2.5.4 Cenozoic Era (65 ± 2 million years B.P. to the present)

2.5.1.2.5.4.1 Tertiary Period (65 ± 2 to 2 ± 1 million years B.P.)

There are no Tertiary age deposits at the site. This was probably a period of erosion in the site area.

2.5.1.2.5.4.2 Quaternary Period (2 ± 1 million years B.P. to the present)

Continental glaciers advanced over the site area several times during the Quaternary Period. Initial deposits in the site area are believed to be associated with the Kansan Stage (called pre-Illinoian Stage by Boellstorf, Reference 4), and consist of clean, fine to medium sand which accumulated in erosional valleys formed on the irregular Pennsylvanian bedrock surface. These deposits are best illustrated in a subsurface section in Figure 2.5-279. The accumulations of sand in the bedrock valleys may have been the result of glaciation some distance upstream. The sand deposition was followed by a relatively thin accumulation of fine alluvial or lacustrine soils with some organic debris which also appears to be confined primarily to the bedrock valleys. Glacial ice of Kansan age eventually reached the site area depositing a blanket of glacial till over the entire area. At this time, some preexisting deposits were removed or reworked and the pre-existing erosional valleys were buried, forming a more or less featureless plain.

The interglacial period following the final retreat of the Kansan glaciers is known as the Yarmouthian Stage. During this time the site area may have been covered by shallow lakes. Topography was undoubtedly low with some vegetation in local areas.

The site was reglaciated during the Illinoian Stage. A sequence of glacial tills, designated as the Glasford Formation, were deposited. Glaciolacustrine deposits and discontinuous lenses of sand and gravel may be interbedded within the glacial tills of the Glasford Formation.

Following the final retreat of the Illinoian glaciers, the deposits of the Glasford Formation at the site were subjected to a long interval of weathering and erosion during and possibly after the interglacial interval known as the Sangamonian Stage. A thick soil (the Sangamon Soil) developed in the upper part of the deposits of the Glasford Formation, and these deposits are referred to as the weathered Glasford Formation in the FSAR and as Interglacial Zone, Sangamon Interglacial Zone, or Sangamon Soil Interval in the PSAR.

During the Farmdalian Substage of the Wisconsinan Stage a thin organic silt, the Robein Silt, was deposited in the site area.

Glaciers advanced over the site for the final time during the Woodfordian Substage of the Wisconsinan Stage, attaining their maximum extent shortly to the west of the CPS site. Meltwater from the retreating Woodfordian ice sheet was responsible for erosion throughout the area. The existing Salt Creek probably was formed in this manner. Outwash and alluvial deposits in Salt Creek have probably accumulated since early Woodfordian time. Initially, runoff was relatively heavy and coarse-grained outwash deposits of the Henry Formation accumulated. As runoff decreased, fine-grained silts and clays of the Cahokia Alluvium accumulated along with some organic debris.

The blanket of wind blown silt (loess) which covers the topographic highs of the site area was transported from sources along the major drainages of Illinois.

2.5.1.2.6 Site Groundwater Conditions

Groundwater conditions at the site are discussed in Subsection 2.4.13.

2.5.1.2.7 Geolocic Considerations

Consideration has been given to all aspects of geology relevant to the suitability of the CPS site, including zones of soft or potentially liquefiable soils, karst, tectonic folding and faulting, slope stability, and the effects of man's activities at or in the vicinity of the site including surface or subsurface subsidence.

The subgrades for all Category I structures are described in Subsection 2.5.4. There is no known karst development at the site or in the vicinity of the site.

No evidence for tectonic faulting or Pleistocene or Holocene folding was noted in preliminary investigations or in the excavations for plant structures at the site.

The stability of slopes at the site is discussed in Subsection 2.5.5.

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.

Present and former activities within the site area have included the removal of sand and gravel and the domestic use of groundwater. Sand and gravel production has been limited to surficial mining operations, and thus no hazard is posed to the plant site because of subsidence. There are no large uses of groundwater nor any industrial disposal wells in this area. No surface subsidence or response due to groundwater withdrawals have been reported near the site.

Two oil fields are located within 15 miles of the Clinton Power Station (Figure 2.5-287). The Wapella East field is located approximately 6 miles northwest of the site, and the Parnell field is located approximately 7 miles northeast of the site (Reference 59). Both of these oil fields are located on domal structures along the Downs Anticline (Figure 2.5-285). There have been no instances of uplift, subsidence, or collapse associated with these oil fields, and no hazard is posed to the plant site because of these oil field developments.

Five gas storage projects are located within 35 miles of the Clinton Power Station (Reference 60) (Figure 2.5-287). The Hudson gas storage project is located approximately 27 miles north of the site; the Lexington gas storage project is located approximately 30 miles north of the site, and the Lake Bloomington project is located approximately 34 miles north of the site (Figure 2.5-287). Each of these gas storage projects was developed by the Northern Illinois Gas Company, and for each of these projects, the storage reservoir is the Cambrian age Mt. Simon Sandstone (Reference 60).

The Manlove gas storage project is located approximately 23 miles east-northeast of the site (Figure 2.5-287). This gas storage project is operated by The Peoples Gas, Light, and Coke Company. The storage reservoir is the Cambrian age Mt. Simon Sandstone (Reference 60).

The Lincoln gas storage project is located approximately 30 miles west of the site (Figure 2.5-287). This gas storage project is operated by the Central Illinois Light Company. The storage reservoir is in Silurian dolomite (Reference 60). There have been no instances of uplift, subsidence, or collapse associated with these gas storage projects, and no hazard is anticipated to the plant site because of these gas storage projects.

2.5.2 Vibratory Ground Motion

This section consists of a discussion and evaluation of the seismic and tectonic characteristics of the Clinton Power Station site and the surrounding region, and presents the rationale used to develop the seismic design criteria for the Clinton Power 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 one of the least seismically active areas of the United States. Since this area has been populated for almost 200 years, it is likely that most earthquake events of Intensity VI and all events of Intensity VII or larger on the Modified Mercalli (MM) Scale (Table 2.5-61) which have occurred during this time span have been reported.

One hundred and sixty-four earthquakes greater than Intensity III are known to have occurred within 200 miles of the site. Table 2.5-4 lists all known reported events greater than Intensity III which have occurred between 37° to 45° north latitude and 84° to 93° west longitude and their locations are shown in Figure 2.5-288. The instrumentally determined locations are probably accurate to about $\pm$ 0.1°. The location of older events, not determined instrumentally, may have occurred as much as $\pm$ 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 earthquakes with an Intensity of VIII or greater within 200 miles of the site; the closest Intensity VIII was 250 miles south at Charleston, Missouri, occurring in 1895 (Figure 2.5-426). If a shock of this size had occurred when the region was only sparsely settled, it almost certainly would have been mentioned in private journals or diaries, or preserved in Indian traditions, as had been the case in other regions. The lack of such documentation indicates the absence of significant earthquake activity within 200 miles of the site for a long period of time.

The greatest earthquakes occurring within 200 miles of the site are listed below. These events had epicentral intensities of MM VII.

a. May 26, 1909 - S. Beloit, Illinois;

b. July 18, 1909 - central Illinois (Havana);

c. September 27, 1909 - southeastern Illinois; and

d. November 9, 1968 - southern Illinois.

Isoseismal maps for these events are reproduced on Figures 2.5-289, 2.5-290, 2.5-291, and 2.5-292. These maps are useful since they show the geographic range of the areas of highest intensity. The greatest intensity induced at the Clinton site by these four earthquakes was MM-V. The closest occurrence of an epicentral intensity (MM) VI to the site was at a distance of approximately 100 miles. Therefore, the maximum intensity experienced at the Clinton site from any earthquake occurring within a 200-mile radius of the site was MM-V.

2.5.2.1.2 Distant Events

2.5.2.1.2.1 Central Stable Region

Only one earthquake occurring in the Central Stable Region at distances more than 200 miles from the site has been felt at the site itself. Docekal (Reference 61, Plate 3) shows Intensity I-III (MM) at the site from the March 8, 1937 Anna, Ohio earthquake (Subsection 2.5.2.3.1.11).

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 62, 63, and 64) at a distance of over 250 miles from the site (Figure 2.5-293 and Table 2.5-5).

Over a period of 3 months during 1811-1812, at least 250 minor events and three major separate shocks occurred, the largest of which had an Intensity (MM) of XI-XII (References 65 and 66). 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 "notable vigorous" shocks occurred in 1776, 1791 or 1792, 1795, 1796, and 1804. Indian traditions also record a previous earthquake which devastated the same area (Reference 65).

In addition to these events, an Intensity (MM) VIII event occurred in 1895 in Charleston, Missouri also within the Mississippi Embayment area, which was probably felt at the Clinton site approximately 250 miles away.

2.5.2.1.2.3 Other Events

One other event was probably felt at the Clinton Power Station site: the 1886 Intensity (MM) X Charleston, South Carolina event which occurred in the Atlantic Coastal Province. Details of these and other distant events are presented in Table 2.5-5. See Dutton, C. E. (1888), The Charleston Earthquake, U. S. Geological Survey 9th Annual Report, Pages 203-528.

2.5.2.2 Geologic Structures and Tectonic Activity

The Clinton site and the vast majority of the 200-mile radius site region lie within the Central Stable Region of the North American Continent (Reference 63). 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 during the Paleozoic resulting in a series of gentle basins, domes, and other structures. Since the end of the Paleozoic, the area has remained generally quiescent. A few square miles of the southernmost area of the site region overlaps the Mississippi Embayment region of the Gulf Coastal Plain Tectonic Province (Reference 63).

The site is located within the Illinois Basin. The most significant nearby structure is the Downs anticline which is genetically related to the La Salle Anticlinal Belt. A description of the faulting and tectonic features in the area is presented in Subsections 2.5.1.1, 2.5.1.2, and 2.5.3.

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 Clinton site, two different approaches were utilized to correlate earthquake activity with geologic structures and/or tectonic provinces: (1) the 200-mile radius site region was subdivided into seismotectonic regions (Subsection 2.5.2.3.1) utilizing methods similar to Stearns and Wilson (Reference 67), and (2) analysis of the relationship of the site to zones of relatively high seismicity within the Central Stable Region Tectonic Province and the Gulf Coast Plain Tectonic Province (Subsection 2.5.2.3.2) were carried out.

2.5.2.3.1 Seismogenic Regions

Eleven seismogenic regions can be delineated within 200 miles of the Clinton Power Station, primarily on the basis of structure. These subdivisions also indicate the differing geologic and seismic histories of the seismogenic regions.

The following subsections describe the eleven seismogenic regions within the 200-mile radius site area and other regions pertinent to the site. Each region is outlined in Figure 2.5-288.

2.5.2.3.1.1 Illinois Basin Seismogenic Region

The site is located in the center of the Illinois Basin Seismogenic Region. The north and northeastern boundaries of this region correspond to and are defined by the limits of the Plum River and Sandwich Fault zones. The region has experienced 60 recorded earthquakes, the largest of which was Intensity (MM) VII. A tentative correlation of some events (with areas of steep gradients in the earth's gravitational field) has been proposed by various authors, notably McGinnis and Ervin (Reference 68). They interpret the gradients as boundaries separating crustal blocks of different densities. However, based on the present state of knowledge, these events are considered random. Therefore, the possibility of an Intensity (MM) VII event anywhere in the basin must be considered.

2.5.2.3.1.2 Ste. Genevieve Seismogenic Region

The Ste. Genevieve Region lies approximately 175 miles southwest of the site and is related to and defined by the imbricated Ste. Genevieve Fault System. This region exhibits a characteristic maximum intensity earthquake of (MM) VI. While there is no geological evidence of capable faulting in this region, fault plane solutions coincide with the trace of the Ste. Genevieve fault. The boundary with the Illinois Basin is based both on a change in structure and by a contrast in seismicity. See Street, R. L. et al., 1974, Earthquake Mechanisms in the Central U. S., Science, V. 184, Pages 1285-1287.

2.5.2.3.1.3 St. Francois Mountains Seismogenic Region

This seismogenic region is defined by the limits of a region of Precambrian rock outcrops in southeast Missouri, constituting the exposed core of the Ozark Uplift. Moderate earthquake activity is associated with faults on the margin of the St. Francois Mountains Regions. The maximum seismic events recorded are Intensity (MM) VI-VII. The closest distance between this region and the site is approximately 175 miles.

2.5.2.3.1.4 Chester-Dupo Seismogenic Region

The Chester-Dupo Region is defined by an area of faulting and folding in the vicinity of St. Louis (Reference 71). This region, approximately 110 miles southwest of the site, is one of moderate seismicity with maximum events characteristic of (MM) VII. The boundary between the Chester-Dupo Seismogenic Region and the Illinois Basin Seismogenic Region is marked by the transition from the folds and faults to the deeper, structurally less complex Illinois Basin. This region marks a hinge line between the Illinois Basin and the northeast boundary of the Ozark Uplift.

2.5.2.3.1.5 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 70 miles This area has moderate seismicity with maximum events of the (MM) VII. Events in this region occur more frequently than events in the adjoining parts of the Illinois Basin (Reference 62). The boundaries of the Wabash Valley Seismogenic Region are well defined by structure and geological history in addition to its seismic pattern.

2.5.2.3.1.6 Western Kentucky Fault Zone Seismogenic Region

This region, 175 miles from the Clinton site, consists of a fault system bending north 80° east. The western boundary with the Illinois Basin Seismogenic Region and the Ste. Genevieve Seismogenic region is defined by a change in seismicity. The southern boundary of the region lies along the boundaries of the New Madrid Region and East Mississippi Embayment Regions and along the northern flank of the Tennessee-Kentucky Stable Region. The Western Kentucky Fault Zone is a stable area with only a few randomly occurring epicenters (Reference 67).

2.5.2.3.1.7 Iowa-Minnesota Stable Seismogenic Region

This region is one of extremely low seismicity with a general maximum intensity of (MM) V. The boundary between this region and the Illinois Basin is approximately 120 miles from the site and is marked by a gentle zone of flexure known as the Mississippi River Arch.

2.5.2.3.1.8 Missouri Random Seismogenic Region

The Missouri Random Seismogenic 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 100 miles southwest of the site. This area is characterized by the occurrence of random seismic events of maximum (MM) V which are not associated with any known structure.

2.5.2.3.1.9 Michigan Basin Seismogenic Region

The Michigan Basin Seismogenic Region is an area of extremely low seismicity with a total of 10 recorded events. The largest seismic event was an (MM) VI.

This area is separated from the Illinois Basin by the Kankakee Arch and lies approximately 150 miles northeast of the site.

2.5.2.3.1.10 Eastern Interior Arch System Seismogenic Region

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, Jessamine Dome, and Wisconsin Dome. 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. Inasmuch as 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 bears more similarity to the Laurentian Shield subdivision of the Central Stable Region of Canada than to the Interior Lowlands subdivision within the United States (References 63 and 64). 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 (MM) V.

The Wisconsin Arch is defined structurally by the low, north south trending, uplifted area extending south from the Wisconsin Dome and is herein defined to include the east-west trending cross-cutting 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 northwest 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 on the bordering Michigan and Illinois 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 (MM) V. However, isolated events of (MM) VII have occurred which cannot be related to specific structures. Therefore, the entire region must be assigned a maximum potential random event of (MM) VII.

2.5.2.3.1.11 Anna Seismogenic Region

The Anna Region lies at the intersection of the Kankakee, Findlay, and Cincinnati arches in western Ohio, 220 miles east of the site. This area has experienced continued and moderately severe seismic activity. The largest historic earthquakes commonly have been of Intensity (MM) VII, with a single event on March 8, 1937, which has been assigned a maximum Intensity (MM) VII-VIII by Coffman and Von Hake (Reference 72). However, a detailed analysis indicates that this event should be reclassified as maximum Intensity (MM) VII (Reference 69).

The Anna 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 69 and 70). The combination of geological features within this area is unique. There is no other known 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 69).

2.5.2.3.1.12 New Madrid Seismogenic Region

The New Madrid Seismogenic Region 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 (References 62, 63, and 64, and Figures 2.5-288 and 2.5-294).

The New Madrid events of 1811-1812 were the largest earthquakes ever experienced in the central and eastern United States. Chimneys were dislodged as far north as St. Louis, Missouri and the aftershocks from these events continued for 2 years (Reference 73). These events occurred more than 260 miles from the Clinton 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 report, dated May 23, 1975, entitled, "Supplemental Discussion Concerning the Limit of the Northern Extent of Large Intensity Earthquakes Similar to the New Madrid Events" (Reference 74). 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. 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° north, 89.2° west - 200 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. These conclusions were also presented to the NRC (formerly the AEC) previously at:

a. AEC staff review meeting in Bethesda, Maryland, for the Clinton Power Station, on June 17, 1974.

b. ACRS Subcommittee Hearings in Urbana, Illinois, for the Clinton Power Station, on March 29, 1975.

c. ACRS Subcommittee Hearing in Bethesda, Maryland, for the Clinton Power Station, on April 4, 1975.

d. ACRS Subcommittee Hearing in Madison, Indiana, for the Marble Hill Nuclear Generating Station, on October 1, 1976.

e. ACRS Full Committee meeting in Washington, D.C., for the Marble Hill Nuclear Generating Station, on October 14, 1976.

A regional microearthquake detection 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 probably two distinct seismic regions (Reference 75).

While all evidence indicates that a New Madrid-type event could only occur in the area of the Pascola Arch, if the events are transposed northward, the major crustal discontinuity along the Rough Creek Fault Zone serves as a boundary for further northward migration. Even if this zone is selected as the northern boundary, the New Madrid-type events could occur no closer than 170 miles from the site.

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 (MM) 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 2.5-294).

The Attica events are associated with the Clarindon-Lindon Structure and the August 12, 1929 event has been assigned an Intensity (MM) VIII by Coffman and von Hake. However, the amount of damage and estimated magnitude of this event indicate that it should be reclassified in the record as an Intensity VII (Reference 76).

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 in Subsection 2.5.2.3.1.11 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 (MM) VII-VIII by Coffman and von Hake (Reference 72), has been analyzed and all indications are that this event has a maximum epicentral Intensity (MM) VII (Reference 69).

The Wabash Valley Fault Zone was described in Subsection 2.5.2.3.1.5 and has had maximum recorded seismic activity of Intensity (MM) VII.

Several events of Intensity (MM) VII have occurred in the area of the Nemaha Anticline magnetic high. 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 77).

The activity near St. Louis, Missouri has been assigned to the Chester-Dupo Region documented in the PSAR for the Callaway Plant (Reference 71). Historical activity in this area has attained a maximum of Intensity (MM) VII.

In addition to these areas of the Central Stable Region which have had relatively high seismic activity, an Intensity (MM) VIII event was reported in the Keewenaw Peninsula of Michigan in 1906 (Reference 72). The felt area of the 1906 event was approximately equal to that of an average Intensity (MM) III-IV event (Reference 72). The area of the epicenter is highly faulted, and the areas of damage and perceptibility coincide with areas of mining activity. Smaller events which occurred earlier in the year as well as the larger event in 1906 all appear directly attributable to mining activity (Reference 61).

2.5.2.3.2.2 Gulf Coastal Plain Tectonic Province

The New Madrid events of 1811-1812 occurred in the Gulf Coastal Plain Tectonic Province, not in the Central Stable Region Tectonic Province. These events are associated with a highly complex structural zone near the crest of the Pascola Arch (Subsection 2.5.2.3.1.12).

If these events are translated to the closest approach of this tectonic province to the site, these events could be expected to occur no closer than 195 miles from the site or 65 miles closer to the site than the 1811-1812 events occurred.

2.5.2.3.3 Earthquake Events Significant to the Site

From both types of analysis of the association of earthquakes with structure as described above, the most significant earthquakes in the region are the July 18, 1909, Intensity (MM) VII central Illinois (Havana) earthquake, the May 26, 1909, Intensity (MM) VII northern Illinois earthquake, the September 27, 1909, Intensity (MM) VII southeastern Illinois - southwest Indiana earthquake, the 1968 Intensity (MM) VII southern Illinois earthquake, and the New Madrid earthquakes of 1811-1812. This evaluation is based on epicentral intensity, 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 a repetition of the 1909 Havana Intensity (MM) 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 (MM) VII event would be expected to envelope the motion from a recurrence of a New Madrid-type event at the closest approach of the Mississippi Embayment, a distance of approximately 195 miles from the site (Subsection 2.5.2.6).

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. These data were used in studies of the site dynamic response.

2.5.2.6 Safe Shutdown Earthquake

The recommended safe shutdown earthquake (SSE) was defined as the occurrence of an Intensity (MM) VII event near the site. This near field event can be correlated to a mean horizontal ground acceleration of 0.13g (Reference 78 and Figure 2.5-295). This level of ground motion would be expected to envelope the motion from a recurrence of a New Madrid-type event at the closest approach of the Mississippi Embayment at a distance of 195 miles from the site.

At the time of the review of the construction permit application, the NRC staff insisted that the safe shutdown earthquake be defined as an Intensity (MM) VIII event near the site. The Illinois Power Company considers this position to be extremely conservative and inconsistent with the seismicity and tectonics of the site region. However, in order to expedite licensing, the NRC staff position was adopted. This resulted in a maximum horizontal ground surface acceleration at the site of 0.25g. To provide an additional margin of safety, this value was applied at foundation level in the free field. Utilizing the subsurface properties presented in Subsection 2.5.4.7, the corresponding ground surface acceleration was found to be 0.26g.

The NRC staff also took the position that the 1811-1812 New Madrid-type earthquakes must be considered to occur at 110 miles from the site, near Vincennes, Indiana. In order to expedite licensing, the effects of the occurrence of such an event was evaluated, and it was shown that the motions generated by the distant event were enveloped by the motions caused by the near field earthquake.

The free field ground response spectra prepared in accordance with Regulatory Guide 1.60 for a horizontal ground acceleration of 0.26g are presented in Figure 2.5-296.

SSER 3 concluded that the site specific spectra are roughly equivalent to 0.20g anchored to the Reg Guide 1.60 spectra at the foundation level in the free field is acceptable.

The vertical component of the free field ground surface spectra for the SSE are presented in Figures 3.7-24 to 3.7-27.

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 during the operating life of the facility. As such, the OBE has been selected on the basis of a seismic risk analysis, the results of which are presented in Subsection 2.5.2.7.1.

2.5.2.7.1 Seismic Risk Analysis

A quantitative seismic risk analysis of the site has been carried out using the method developed by Merz and Cornell (Reference 79 (a)). The output of this analysis is a plot of annual risk (probability of exceeding acceleration) versus peak ground acceleration at the site.

The risk analysis is based on the historic seismicity in each of the seismotectonic provinces described in Subsection 2.5.2.3.

The following earthquake sources have been considered in the risk analysis:

a. Illinois Basin Seismogenic Region,

b. Ste. Genevieve Seismogenic Region,

c. St. Francois Mountains Seismogenic Region,

d. Chester-Dupo Seismogenic Region,

e. Wabash Valley Seismogenic Region,

f. Western Kentucky Fault Zone Seismotectonic Region,

g. Iowa-Minnesota Stable Seismogenic Region,

h. Missouri Random Seismogenic Region,

i. Michigan Basin Seismogenic Region,

j. Eastern Interior Arch System Seismogenic Region,

k. Anna Seismogenic Region, and

l. New Madrid Seismogenic Region.

Other seismotectonic provinces discussed in Subsection 2.5.2.3 are either too far from the site or have seismicity too low to require consideration.

The mean annual number of events ($\geq$ MM Intensity IV), known as "activity rates," for the various sources are given in Table 2.5-65. These rates have been obtained by dividing each source's total number of events between 1879 and 1978 by 100. The table also shows the historical maximum Mercalli intensity for each source zone, as discussed in Subsection 2.5.2.3.

The probability distribution of events of different epicentral intensities, I_e, is chosen in the form recommended in Reference 79(b):

$$\ln \{P [I_e \geq i_e]\} = -\beta_1(i_e - i_\ell) \quad \text{for} \quad i_\ell < i_e < i_u \tag{2.5-1}$$

where i_ℓ and i_u are the lower and upper epicentral intensity bounds and β_1 is a shape coefficient which is conservatively estimated as 0.863 (Reference 80). The lower bound intensity for the local earthquake source used in the analysis is (MM) IV; for all other sources the value is (MM) V. The upper bound intensity used in the analysis for each source is shown in Table 2.5-65.

The spatial attenuation of intensity (Equation 2.5-2) in the central United States as developed by Gupta (Reference 81) is considered appropriate for the site region and is used in this analysis where I_s is the site intensity and R is the epicentral distance in kilometers.

$$I_s = I_e + 2.35 - 1.79 \log_{10}R - 0.00316R \tag{2.5-2}$$

for R $\geq$ 20 km. The standard deviation, s, of the error in I_s of this relationship is estimated as 0.45 by the same author. The correlation of site intensity, I_s, and the peak ground acceleration, a, is obtained from Reference 82:

$$\log_{l0}a = 0.24I_s + 0.26 \tag{2.5-3}$$

and the antilog of the standard error of a is 2.19.

For the sources closer than 20 km to the site, the form of Equation (2.5-3) remains as it is since no attenuation of intensity (i.e., $I_s = I_e$) is used for these sources in this study.

For sources farther than 20 km from the site, the acceleration, a, depends on the epicentral intensity and the distance (i.e., by combining Equations (2.5-2) and 2.5-3)):

$$\log_{10}a = 0.24I_e + 0.824 - 0.0007584R - 0.4296\log_{10}R \qquad (2.5\text{-}4)$$

The computer program SRA (Seismic Risk Analysis) developed by MIT and updated by Sargent & Lundy has been employed in this analysis. The concept of risk calculations by SRA can be understood using the following example. The total number of events per year which will cause a site peak ground acceleration greater than or equal to 0.05g is the sum of contributions from individual sources. The latter are the sums of the expected numbers of events from the elemental areas making up the source.

The expected number of events from each elemental area of the source is simply the product of (a) the mean annual activity rate, and (b) the fraction of events which will be large enough to generate a peak ground acceleration ≥ 0.05g at the site. How large such an event must be (i.e., how large its epicentral intensity must be to cause a peak ground acceleration ≥ 0.05g at the site) can be estimated by using the attenuation and intensity-peak ground acceleration relationships. The numerical procedure adopted herein (Reference 79) permits an accounting for the quoted uncertainties in these relationships. Once the required epicentral intensity is known, the fraction of all events that are equal to or greater than that intensity value can be obtained from the epicentral intensity probability distribution.

The seismic risk curve due to all relevant seismic sources is shown in Figure 2.5-427. It shows the resulting annual risk that any specified peak horizontal ground acceleration will be exceeded. The operating basis earthquake for the plant has been selected as 0.10g applied at the foundation level. Utilizing the subsurface properties presented in Subsection 2.5.4.7, the corresponding ground surface acceleration was determined to be 0.11g. From Figure 2.5-427, it can be seen that the annual probability of exceeding this OBE acceleration at the site is 1.5×10^{-3} per year (or mean recurrence interval of about 650 years). Based on this low probability, it is concluded that the selection of the OBE has been conservative.

The free field ground response spectra prepared in accordance with Regulatory Guide 1.60 for a horizontal ground surface acceleration of 0.11g are presented in Figure 2.5-297.

The vertical component of the free field ground surface spectra for the OBE are presented in Figures 3.7-16 to 3.7-19.

2.5.3 Surface Faulting

There is no evidence for surface faulting at the site or the area surrounding the site (200-mile radius around the plant site). Further, faults which have been mapped in Illinois have shown no sign of movement during Quaternary time.

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, as defined by 10 CFR 100 Appendix A, within 200 miles of the site.

The closest proposed fault to the site is the so-called Tuscola Fault (Reference 47), postulated to trend north-south, approximately 20 miles east of the Clinton Power Station. The existence of this fault has not been accepted by the Illinois State Geological Survey (Reference 48). The nearest confirmed fault is the Sandwich Fault Zone, located approximately 90 miles northeast of the Clinton Power Station. The last movement on the Sandwich Fault Zone occurred during the interval from Post-Silurian to Pre-Pleistocene time, probably in the late Paleozoic (Reference 22).

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 or near the site. The structural geology at the site and surrounding region is discussed in Subsections 2.5.1.1.4 and 2.5.1.2.3.

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 of the site.

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

Geologic investigations of the site have not found any evidence of capable faulting; therefore, the detailed fault investigation required for capable faulting 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; therefore, a study of surface faulting is not required.

2.5.4 Stability of Subsurface Materials and Foundations

This section presents the evaluation of the stability of subsurface materials that underlie the site. This evaluation is based upon preliminary soils studies conducted prior to construction operations and the properties of the soils encountered during the earthwork construction.

2.5.4.1 Geologic Features

A detailed discussion of the geologic characteristics of the site is given in Subsection 2.5.1.2.

2.5.4.2 Properties of Subsurface Materials

This subsection presents static and dynamic engineering properties of the subsurface materials encountered in the borings drilled as part of the investigation program, the properties of the materials used during the site excavation, and the properties of the materials used as structural fill and backfill. The material properties were based upon:

a. a review of all available 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,

d. a review of similar studies performed recently for nuclear generating stations at other locations, and

e. testing performed during construction.

Representative soil samples and rock cores extracted from the test borings were subjected to laboratory tests to evaluate the physical and chemical characteristics of the soil and rock encountered at the site. The physical and chemical characteristics of the groundwater encountered at the site were evaluated by testing representative samples obtained from test borings and piezometers at the site. The laboratory program for the PSAR stage included the following tests:

a. strength test -

 1. direct shear,

 2. unconfined compression on undisturbed samples,

 3. unconfined comppression on remolded samples,

 4. unconsolidated-undrained triaxial compression, and

 5. consolidated-undrained triaxial compression - some with pore pressure measurements;

b. dynamic tests -

 1. cyclic triaxial,

 2. resonant column, and

 3. shockscope;

c. other physical tests -

1. Atterberg limit,

2. consolidation,

3. in situ moisture and density determinations,

4. permeability, and

5. relative density;

d. chemical tests on groundwater samples; and

e. chemical analysis on coal samples.

The results of the unconfined compression, unconsolidated-undrained triaxial, consolidated-undrained triaxial, moisture and density, and Atterberg limit tests are presented to the left of the logs of borings. Tests performed and reported elsewhere are indicated by symbols in the left-hand column of the logs of borings. The key to the test symbols is presented at the bottom of Figure 2.5-298.

Subsection 2.5.4.2.6 presents soil investigations and testing programs undertaken, following the submittal of the PSAR, on in situ material as well as fill material under the guidance of Sargent & Lundy personnel as well as the Quality Control Program instituted at the CPS site during the construction of the power station. These programs were initiated to confirm soil properties determined in initial investigations and used in the design considerations for the earthwork construction.

2.5.4.2.1 Strength Tests

2.5.4.2.1.1 Strength Tests on Soil

Selected representative soil samples were tested in the manner described in Figure 2.5-299 to determine their strength characteristics.

Unconsolidated-undrained triaxial compression and unconfined compression tests were performed on selected, undisturbed samples to evaluate their undrained strength characteristics. Consolidated-undrained triaxial, with and without pore pressure measurements, and direct shear tests were performed on samples to evaluate the effective stress parameters. The samples tested were sheared under a surcharge or confining pressure corresponding approximately to the effective soil and/or surface loads occurring, or expected to occur, in the field. The triaxial and unconfined compression tests were run in the manner shown in Figure 2.5-299. The direct shear tests were run in the manner shown in Figure 2.5-300.

A load-deflection curve was plotted for each test and the strength of the soil was determined from this curve. Determinations of the field moisture content and dry density of the soil were made in conjunction with each strength test. The results of the strength tests and the corresponding moisture content and dry density determination are presented to the left of the boring logs (Figures 2.5-19 through 2.5-270) and in Tables 2.5-6 through 2.5-17.

2.5.4.2.1.2 Strength Tests on Rock

The strengths of the underlying rock formations were evaluated by subjecting representative rock core sampling to unconfined compression tests (ASTM D2938-71). The tests were performed by subjecting samples approximately 4 inches in height and 2 inches in diameter to a constant rate of axial load. The modulus of elasticity of the rock was evaluated by recording the deformation of the rock and computing the stress-strain relationship. The results of the rock compression tests are presented to the left of the boring logs.

2.5.4.2.2 Dynamic Tests

2.5.4.2.2.1 Cyclic Triaxial Tests

The behavior of representative soils under dynamic loading was evaluated by conducting dynamic triaxial tests in the manner indicated in Figure 2.5-301. The samples were initially allowed to consolidate under confining pressures representative of existing in situ conditions. Strain controlled dynamic triaxial tests were performed on the consolidated samples by subjecting them to sinusoidally varying axial strains at a frequency of 1 hertz. From these tests dynamic material properties such as strain dependent shear modulus and damping values were obtained. The test results are presented in Tables 2.5-18 through 2.5-25.

Dynamic triaxial tests were performed on remolded samples of material similar to that used as structural fill beneath the plant and the test results are shown in Figures 2.5-302 through 2.5-311. Dynamic triaxial tests were also performed on representative in situ soils obtained from borings in the ultimate heat sink area. The results of these tests are shown in Figures 2.5-312 through 2.5-317 and in Table 2.5-22.

2.5.4.2.2.2 Resonant Column Tests

Dynamic torsional shear (resonant column) tests were performed on representative soil and rock samples to evaluate the modulus or rigidity (shear modulus) of these materials in the manner described in Figure 2.5-318. The tests were conducted at natural moisture content over a range of confining pressures. The results of the resonant column tests are presented in Tables 2.5-26 through 2.5-29.

2.5.4.2.2.3 Shockscope Tests

Compressional wave velocity (shockscope) tests were performed on representative soil and rock samples. The velocity observed in the laboratory was used to correlate with field velocity measurements obtained during the geophysical survey.

In this test, samples are subjected to a physical shock, and the time required for the shock wave to travel the length of the sample is measured. The velocity of compressional wave propagation is then computed. The samples were tested in an unconfined state. The test results are presented in Table 2.5-30.

2.5.4.2.3 Other Physical Tests

2.5.4.2.3.1 Atterberg Limit Tests

Representative soil samples were tested to evaluate their plasticity characteristics (ASTM D424-59 and ASTM D423-66). The results of these tests were used for classification and correlation purposes. The Atterberg limit determinations are presented to the left of the boring logs.

2.5.4.2.3.2 Consolidation Tests

Consolidation tests were performed on representative cohesive soil samples and granular borrow materials to determine compressibility characteristics of the soils. The tests were run in the manner described in Figure 2.5-319. Tests performed on Pitcher samples and the 4-inch core samples (high recovery core barrel method of sampling) were run using a procedure suggested by Bjerrum (Reference 83). The test results are presented in Figures 2.5-320 through 2.5-346.

2.5.4.2.3.3 In Situ Moisture and Density Determinations

In addition to the in situ moisture and density determinations made in conjunction with the previous tests, additional moisture 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 of the boring logs. The moisture content determinations were performed according to ASTM D2216-66.

2.5.4.2.3.4 Laboratory Permeability Tests

Falling-head and constant-head type permeability tests were performed on representative soil samples to provide data for determining groundwater movements in the vicinity of the CPS site as well as in the vicinity of the main dam. The tests were performed in the manner described in Figure 2.5-347. The results of these tests are presented in Tables 2.5-31, 2.5-32, and 2.5-33.

2.5.4.2.3.5 Relative Density Tests

Relative density tests were performed on selected, representative samples from the Mahomet Bedrock Valley deposit to determine the minimum and maximum densities. The results of these tests are presented in Table 2.5-34. These tests were performed according to ASTM D2049-69.

2.5.4.2.4 Chemical Tests

Chemical analysis was performed on representative groundwater samples obtained from selected borings. The test results are presented in Subsection 2.4.13.1.3. The methods used were generally those given in "Standard Methods for the Examination of Water and Wastewater," Thirteenth Edition, 1971 (Reference 84). Calcium and magnesium were determined by atomic absorption.

2.5.4.2.5 Chemical Analysis on Coal Samples

Chemical analysis was performed on coal samples for the No. 8 and No. 7 coal beds in Boring P-38. The analysis was performed by the Illinois State Geological Survey and is presented in Table 2.5-3.

2.5.4.2.6 Construction Stage Investigations

This subsection presents static and dynamic engineering properties of the underlying materials encountered in the additional borings drilled and representative bulk samples taken as part of the engineering investigations and quality control program at the CPS site following the submittal of the PSAR.

During the earthwork construction at the project site, there was a continuous program of monitoring the quality control of the earthwork construction and monitoring the soil conditions to confirm conformance with the design conditions. These monitoring activities involved boring programs, sampling programs, and testing programs. The data obtained from these programs provided documentation of the conformance of the work with the project earthwork specifications.

Representative soil samples extracted from the test borings and stockpiles of borrow materials were subjected to laboratory tests to evaluate and document the physical characteristics of the in situ soils as well as the material placed as fill and backfill. The laboratory programs included the following tests:

a. strength tests -

   1. unconfined compression on disturbed and undisturbed samples;

b. dynamic tests -

   1. liquefaction tests;

c. other physical tests -

   1. Atterberg limit,

   2. compaction,

   3. in situ moisture and density determinations,

   4. particle size analysis including hydrometer, and

   5. relative density.

The borings drilled after the writing of the PSAR are identified on the logs of borings as being logged by Sargent & Lundy. The results of the unconfined compression, in situ moisture and density, and Atterberg limit tests are presented to the left of the logs of borings. Other tests performed are indicated by symbols in the left-hand column of the logs of borings. The key to the test symbols is presented at the bottom of Figure 2.5-298.

2.5.4.2.6.1 Dynamic Tests

2.5.4.2.6.1.1 Laboratory Liquefaction Tests

A series of laboratory liquefaction tests was performed on the Type B granular structural fill borrow material. Type B material is described in Subsection 2.5.4.5.1.5. Four bulk samples were taken from the material stockpile used in the construction operation in order to have representative samples that closely correlate with the actual material used. The grain size characteristics are presented in Figure 2.5-348.

To prepare the samples for testing, a preweighted amount of moist material with a water content of 7% was placed into a membrane lined forming mold 2.4 inches in diameter and 6 inches high in six equal layers. Each layer was compacted by a combination of vibratory and hand tamping methods to achieve the specified density. The densities of the samples ranged from 126.4 pcf to 127.2 pcf dry density. These values correspond to the average dry density at 85% relative density.

Carbon dioxide gas was then applied through the specimen for a period of time to aid in evacuating all of the air out of the specimen. Slight vacuum was then applied to maintain the shape of the specimen as the mold was removed and while micrometer measurements of the specimen diameter and height were obtained. The triaxial cell was assembled around the specimen, a small confining pressure applied, and the vacuum was released allowing water to slowly flow through the sample until it was nearly saturated. To fully saturate the specimen, back pressure was applied to the specimen through a water chamber with a confining pressure slightly higher than back pressure. The required effective confining pressure was then applied and testing was performed with values of Skempton's pore pressure parameter B exceeding 0.95.

The liquefaction tests were performed under controlled stress conditions using a pneumatic sinusoidal wave loading system. After the specimen was fully consolidated, the drainage valve was closed and the specimen was subjected to a constant amplitude sinusoidal cyclic vertical load at a frequency of approximately 1 hertz.

The traces of load, deformation and the pore water pressure with time were recorded on a strip chart recorder during testing.

The criterion for liquefaction of the laboratory samples was defined as the number of cycles required to produce initial liquefaction. Initial liquefaction is defined as when the pore pressure first becomes equal to the effective confining pressure. For the Type B structural fill, the 5% double amplitude strain and the initial liquefaction curves are shown in Figure 2.5-349. During cyclic testing, it was noted that the axial strain developed mostly at a small horizontal zone near the top portion of the specimen. It should be noted that both the recorded pore pressures and axial strains during cyclic loading cannot be attributed to uniform behavior of the entire specimen. Therefore, data points shown in Figure 2.5-349 for initial liquefaction and for 5% double amplitude strain obtained during cyclic tests are conservative. A summary of the soil properties and the liquefaction test results is presented in Table 2.5-35. The results of the liquefaction tests are shown in the form of stress ratio (ratio of one-half of the cyclic vertical stress to confining pressure) versus the number of stress cycles required to produce liquefaction (or specified strain levels).

2.5.4.2.6.2 <u>Other Physical Tests</u>

A laboratory testing program was established at the site to verify that the materials used during construction as structural fill and backfill met the specifications established for the materials. The following tests were performed on representative samples to maintain controls on the materials used and to ensure that the placed material conformed to the design considerations.

2.5.4.2.6.2.1 Atterberg Limit Tests

Representative soil samples were tested to evaluate their plasticity characteristics (ASTM D424-59 and ASTM D423-66). The results of these tests were used for classification and correlation purposes to verify the uniformity of the material. The Atterberg limit determinations are presented to the left of the boring logs. Table 2.5-37 includes a summary of the range of values for the Atterberg limit of the compacted cohesive material that was placed as fill material in the main plant and screen house areas.

2.5.4.2.6.2.2 Compaction Tests

Compaction tests were performed on representative bulk samples of Wisconsinan till used as structural fill and backfill to maintain and verify the uniformity of the material. The compaction tests were performed in accordance with the American Society for Testing and Materials (ASTM) Test D1557. The results of the tests are presented in Figure 2.5-350 for material used as backfill in the plant and screen house areas.

2.5.4.2.6.2.3 In Situ Moisture and Density Determinations

In situ moisture and density tests were performed on soil samples taken during drilling operations. The results of these moisture and density determinations are presented to the left of the boring logs. The moisture content determinations were performed according to ASTM D2216-66.

In situ moisture (ASTM D-2216 and D-3017) and density (ASTM D-1556, D-2922, and D-2167) tests were also performed on the material placed as structural fill and backfill as part of the quality control program at the site. This is discussed in Subsection 2.5.4.5.

2.5.4.2.6.2.4 Particle Size Analyses

Particle size distributions were determined for representative soil samples of both the fill materials and various soil strata to aid in classification and correlation of the physical soil properties as well as to verify the conformity of its gradation with the specifications. Figure 2.5-351 presents the statistical average grain-size distribution of the Type B materials used for structural fill and backfill in the plant area. These tests were performed according to ASTM D422-63.

2.5.4.2.6.2.5 Relative Density Tests

Relative density tests were performed on selected, representative samples of coarse-grained soils to determine the minimum and maximum densities for placement of the material as structural fill and backfill. The results of these tests are presented in Figure 2.5-377. These tests were performed according to ASTM D2049-69.

2.5.4.3 Exploration

The subsurface soil, rock, and groundwater conditions at the station site (76 borings), at the ultimate heat sink (60 borings), at the dam site (60 borings), at the dam borrow area (17 borings), at the location of Section E-E' along the North Fork of Salt Creek (4 borings), and at the structural fill borrow areas (31 borings), were explored by drilling test borings at the locations indicated in Figures 2.5-14, 2.5-16, 2.5-271, 2.5-272, 2.5-273, 2.5-352, and 2.5-353.

The borings taken in the PSAR stage were drilled with truck-mounted rotary wash or continuous-flight auger equipment by Raymond International, Inc., under the supervision of Dames & Moore. The rock was cored utilizing NX double-tube core barrels, which provide rock cores approximately 2 inches in diameter. Undisturbed soil samples suitable for laboratory testing were obtained using Dames & Moore Type U Sampler, a Pitcher sampler, and a 4-inch inside diameter double-tube core barrel (High Recovery Core Barrel). The Dames & Moore sampler is illustrated in Figure 2.5-354. This sampler is 3-1/4 inches in outside diameter and approximately 2-1/2 inches in inside diameter. The Pitcher sampler consists of a stationary thin inner barrel and a rotating outer barrel with a cutting bit, which is drilled into the soil. The stationary inner barrel has an outside diameter of 3.0 inches and an inside diameter of approximately 2.9 inches. The 4-inch inside diameter double-tube core barrel is very similar to the conventional double-tube rock core barrel. The core bit has an inside diameter of 4.0 inches and an outside diameter of 5.5 inches. Disturbed soil samples were extracted utilizing a standard split-spoon samplier approximately 2 inches in outside diameter. These samples were taken using the Standard Penetration Test procedure (ASTM D-1586).

Additional boring programs were undertaken under the guidance of Sargent & Lundy personnel. The borings were drilled with truck-mounted rotary wash or continuous-flight auger equipment by Raymond International, Inc. Samples obtained during these programs were extracted utilizing a standard split-spoon sampler approximately 2 inches in outside diameter. These samples were taken using the Standard Penetration Test procedure (ASTM D-1586). Other sampling methods and equipment used included the 4-inch inside diameter double-tube core barrel, Pitcher sampler, Osterberg sampler, and Shelby tubes as described above. The borings taken for these programs are so noted on the logs themselves.

A graphical representation of the soils and rock encountered in the borings, including standard penetration tests data, sampling, and coring information, is presented in Figures 2.5-19 through 2.5-270 and 2.5-439 through 2.5-450. The method utilized in classifying the soils and rock is described in Figure 2.5-355. The plans and profiles for the main plant excavations are discussed in Subsection 2.5.4.5. Attachment C2.5 presents the geologic mapping performed at the CPS site.

A key to the sample symbols and the sampling information presented on the log of borings is shown in Figure 2.5-298.

Piezometers were installed in the boreholes to observe ground-water conditions. They consist of 3/4-inch PVC pipe having an 18-inch-long porous stone at the bottom. A bentonite plug was installed above the porous stone which was enclosed in granular backfill. When the intended zone of percolation to be measured was large, 3/4-inch-to-2-inch-diameter perforated PVC pipe without a porous stone tip was used as the groundwater observation well.

Nine additional piezometers were installed downstream of the main dam to monitor groundwater and seepage. They consisted of 1-1/2-inch diameter PVC pipe with the lower end plugged and the lower 5 feet slotted. A bentonite plug was installed above the slotted zone which was enclosed in granular backfill. A summary of the elevations at which piezometers were installed and observations of water levels are presented in Subsection 2.4.13.2.3.

Falling-head-type field permeability tests were performed at the dam site and the CPS site. The results of these tests are presented in Table 2.5-38.

2.5.4.4 Geophysical Surveys

A program of integrated geophysical explorations was conducted at the station site, the dam site, and at Section E-E' along the North Fork of Salt Creek. This program consisted of the following field studies:

a. a seismic refraction survey to evaluate the compressional wave velocities of bedrock and the materials overlying bedrock (results of this survey were also used to provide additional data to determine the depth to bedrock under the site);

b. an uphole velocity survey to further define the compressional wave velocities of the materials overlying bedrock;

c. a surface wave survey to determine surface wave types and characteristics;

d. a downhole shear wave survey to evaluate the shear wave velocities of bedrock and of the materials overlying bedrock; and

e. ambient noise studies to determine the predominant characteristics of ground motion due to background noise levels.

The locations of these field studies are shown in Figures 2.5-356 through 2.5-358 and Figure 2.5-16.

2.5.4.4.1 Seismic Refraction Survey

A seismic refraction survey was conducted by Dames & Moore at the station site along five seismic test lines for the total length of 6100 lineal feet. In addition, seismic refraction surveys were conducted at the dam site, and at Section E-E' along the North Fork of Salt Creek. The total length of these two additional surveys was 4000 lineal feet.

The length of seismic test lines in the station site area was limited due to the proximity of two oil-product pipelines which pass through the area just south of the station site.

The seismic energy used in the survey was produced by explosive charges placed in drilled holes. These holes ranged in depth from 10 to 25 feet. The explosive charges ranged in size from 5 to 15 pounds of Nitramon-S (Du Pont).

The energy released by the explosives was picked up on vertically oriented geophones spaced at 100-foot intervals along the seismic test lines. The geophones, manufactured by Electro-Tech Labs, have a natural frequency of 14 cycles per second (Hertz) and are fitted with a spike to ensure proper coupling with the underlying soil.

The energy impulse picked up by the geophones was recorded by an Electro-Tech Labs M4-E seismic amplifier coupled with an SDW-100 recording oscillograph. An Electro-Tech Labs ER-75-12A seismic recording amplifier was also used to record the energy impulses.

The geophysical field crew consisted of two geophysicists, a licensed powderman with helper, and a driller with helper. The field work was performed from June 6 to June 23, 1972.

The compressional wave velocities and the corresponding depths to the various subsurface layers under the site were evaluated by plotting the first arrival times of the seismic energy at each geophone against the distance of each geophone from the source of the seismic energy. The time-distance data from each profile and the corresponding cross section of the subsurface layers for that profile are shown in Figures 2.5-359 through 2.5-365.

2.5.4.4.2 Uphole Velocity Survey

An uphole velocity survey was performed by Dames & Moore at the CPS site, at the dam site, and at Section E-E' along the North Fork of Salt Creek. This provided a check on the compressional wave velocities measured during the seismic refraction surveys at each of these locations.

Explosive charges of 1 pound of Nitramon-S were placed in drill holes to depths of 5 feet. These drill holes were offset from the boring to be studied by a distance of 25 feet. The seismic energy released by the explosives was detected in the boring by twelve geophones affixed to a special cable (velocity cable). The energy was recorded on an Electro-Tech Labs ER-75-12A seismic recording amplifier.

The results of the uphole velocity surveys are presented in Figures 2.5-366 through 2.5-368.

The variation between the compressional wave velocities as determined by the uphole method and the refraction method is a result of the different techniques used in these methods, and the differences in the wave paths travelled by the seismic energy. The seismic refraction survey measures the compressional wave velocities over a lateral distance, whereas the uphole velocity survey measures the compressional wave velocities around an isolated point (the boring).

The uphole survey for Boring P-14 has not been grossly averaged. All points as shown are within a 2 msec deviation from the average velocity curve. A 2 msec deviation is considered normal, in that a 1 msec tolerance is allowed for the picking of the timebreak, and a 1 msec tolerance is allowed for the picking of the energy arrival.

The interval in Boring P-14 from approximately 10 to 60 feet below the ground surface shows the largest deviation of the compressional wave velocities between these two methods (uphole and refraction). This is a result of the material within that layer. This material could show a slight increase in velocity with depth. However, because of the ± 2 msec error factor, this magnitude of velocity change in the uphole survey would not be accurately differentiated. The refraction work resulted in showing the higher velocity.

A 100-foot geophone spacing is not too large to capture the effects of layering. The seismic refraction method is based on the compressional wave velocities and corresponding thicknesses of different layers. If a sufficient velocity contrast between two layers does not exist, if the layers are not sufficiently thick, or if velocities decline with depth, no refraction of a compressional wave will be noted. The uphole survey was performed on the site before the refraction survey. The interpretation of the uphole survey did not indicate any velocity inversions or sufficient velocity contrasts or layering thicknesses other than as shown. Therefore, the 100-foot geophone spacing was selected for the refraction work.

Velocity measurements, other than those shown, cannot be determined for additional layers from the uphole survey. A common technique used in the petroleum exploration industry is to integrate sections of an uphole velocity survey to obtain interval compressional wave velocities for different units. This technique is not possible, however, with the accuracy involved (± 2 msec) for the uphole survey in Boring P-14.

The geophysical program was established to determine the elevation of the bedrock beneath the site. The length of the refraction lines used for this purpose was not great enough to allow the delineation of any other major velocity units beneath the top of bedrock.

The depth of the high velocity rock (velocity range from 9750 to 10,500 feet per second) in the station area is shown on the subsurface sections accompanying each time-distance plot.

2.5.4.4.3 Surface Wave Survey

Surface wave surveys were conducted by Dames & Moore along a 2000-foot line in the CPS area, an 1800-foot line at the dam site, and a 1300-foot line at Section E-E' along the North Fork of Salt Creek.

The surface wave characteristics and shear wave velocities were computed from the recordings of two three-component geophones (Sprengnether Engineering Seismograph geophones) and eight one-component geophones. The Sprengnether geophones were placed 300 to 350 feet apart, and the one-component geophones were placed at 50-foot intervals between the Sprengnether geophones. The output of all the geophones was recorded by an Electro-Tech Labs M4-E seismic amplifier and an SDW-100 recording oscillograph.

The seismic energy for this survey was produced by small explosive charges detonated from 400 to 2000 feet from the geophone array.

The characteristics of the surface waves for each site are listed in Tables 2.5-39 through 2.5-41.

The characteristic frequency range is between 7 and 12 hertz. Significant amplification of seismic energy will probably occur only with this frequency range.

It was also observed that the vertical component of particle motion attenuates much faster at the dam site than at the station site. This is probably a result of the different site conditions (subsurface) and the difference in the site geometry between the station site and the dam site.

The surface wave survey provided data on shear wave velocities of the near-surface materials. These velocities confirmed the shear wave velocities measured by the downhole techniques.

2.5.4.4.4 Downhole Shear Wave Survey

A downhole shear wave survey was performed by Dames & Moore at each site utilizing borings that had been drilled into bedrock. Two three-component low-frequency geophones (Mark Products L-1-3DS) were placed at different depths in each boring.

Small explosive charges were detonated at varying distances from each boring. The resultant seismic energy was recorded by an Electro-Tech Labs M4-E seismic amplifier and an SDW-100 recording oscillograph. In addition, the seismic energy resulting from a technique of producing horizontally generated waves was also recorded. Recordings were made in each boring at successive 25-foot intervals.

Shear wave arrivals are often masked by the compressional wave energy or the relatively large amplitude motion induced by surface and body wave systems. To overcome this difficulty, high-energy and low-energy recordings were made at each depth in the borings.

The results of the downhole shear wave values and surface wave shear wave values are presented in Figures 2.5-369 through 2.5-371. These data are referenced to the subsurface conditions found at each site.

2.5.4.4.5 Ambient Vibration Measurements

Measurements of the levels of ground motion due to background (ambient) vibrations were taken at each site at the locations shown in Figures 2.5-356 through 2.5-358. The measurements were taken when no equipment or drills were operating.

A three-component VS-1200 Sprengnether Engineering Seismograph was used to record ambient ground motions. This seismograph has gain characteristics in the velocity mode of 20 inches/inch/second, the acceleration mode of 12 inches/inch/second/second, and the displacement mode of 200 inches/inch. A VS-1100 amplifier with a gain characteristic of 100 was used for all recordings. The resultant maximum gain level for the velocity mode is 2000; for the acceleration mode, 1200; and for the displacement mode, 20,000.

The three components of ground motion measured were radial, vertical, and transverse. The seismometer was oriented facing north.

The results of the ambient ground motion measurements are presented in Table 2.5-42.

2.5.4.4.6 Summary of Laboratory and Field Wave Velocity Measurements

Compressional wave velocities derived from the laboratory dynamic triaxial tests and shockscope tests are summarized in Table 2.5-43. Shear wave velocities derived from the laboratory resonant column tests are shown in Table 2.5-44. Compressional and shear wave velocities derived from the geophysical field data are summarized in Tables 2.5-45 and 2.5-46, respectively.

Comparing the laboratory and field wave velocity data shows that field and laboratory wave velocities are significantly different from each other. These differences occur because dynamic properties of soil and rock are strain-dependent. The laboratory dynamic triaxial tests and resonant column tests are usually performed with 5 to 10^{-2} percent and 10^{-2} to 10^{-4} percent of shear strain, respectively; whereas the field geophysical methods usually result in shear strains in the range of 10^{-3} to 10^{-5} percent. Actual strong-motion earthquakes generally produce shear strains between 10^{-1} to 10^{-3} percent. Therefore, the response calculations, wave velocities and dynamic properties derived from laboratory triaxial compression and resonant column tests at the shear strain of interest to the prevailing earthquake conditions are the most appropriate to be used.

The shockscope tests were performed at zero confining pressure and at a level of strain considerably lower than most significant earthquakes. The wave velocities data as determined by the shockscope would therefore not be realistic and reliable for analysis.

2.5.4.5 Excavations and Backfill

2.5.4.5.1 Station Site

2.5.4.5.1.1 Site Preparation

Site preparation and earthwork for CPS Units 1 and 2 consisted of stripping, excavating, dewatering, and backfilling operations to attain a nominal station grade at approximately elevation 736 feet.

Trees, brush, crops, grass, roots, and other deleterious materials were stripped from areas to be occupied by structures and from all areas that received fill. All topsoil was removed prior to general excavation operations.

2.5.4.5.1.2 Excavation

The excavation for the main power station was an open excavation performed by heavy earth moving scrapers. The excavation was approximately 800 feet by 800 feet and it extended from existing grade to the Illinoian till of the unaltered Glasford Formation at approximately elevations 680 to 683. The maximum height of the construction slopes was approximately 56 feet in the excavation for the station. The limits of the excavation were extended a minimum of 20 feet beyond the limits of the structures themselves in order to ensure adequate foundation support within the zone of major stress concentrations.

Based on the results of comprehensive engineering analyses, construction slopes for the major excavations were cut to a general slope of 1.5:1 (horizontal to vertical), but no steeper than 1:1 (horizontal to vertical).

Figure 2.5-16 shows the location of Seismic Category I structures and exploration in the vicinity of these structures.

Excavation details with geologic sections are shown in Figures 2.5-372 and 2.5-373. A plan of the station foundations is shown in Figure 2.5-374.

2.5.4.5.1.3 Dewatering

The subsoil and groundwater conditions in the station area were such that no significant dewatering problems occurred. The rate of seepage into the excavations extending below the groundwater level was very low in the natural clayey till soils. However, more previous sand layers and seams did contribute to the rate of seepage.

Dewatering was accomplished by a network of perforated metal pipe drains and ditches that collected the seepage at the periphery of the excavation. The seepage water was then drained from the excavation by gravity flow to the heat sink area to the west of the site. Water was not allowed to pond in the base of the plant excavation to permit base treatment and fill placement under dry conditions.

2.5.4.5.1.4 Excavation Base Treatment

The base of the main plant excavation was on Illinoian till of the unaltered Glasford Formation. Pockets of sand exposed in the base of the excavation were tested as described in the following.

A comprehensive program of subgrade inspection and testing was initiated to verify the adequacy of the subgrade and compare the existing conditions with those postulated from the soil borings in the PSAR stage. Subgrade testing was performed as a part of the contractor's quality control program.

Field testing on subgrade consisted of determining the in situ density using either the sand cone (ASTM D-1556) or nuclear densometer (ASTM D-2922) method. Laboratory testing consisted of the classification of soils by the unified classification method (ASTM D-2487) and determination of index properties using the applicable ASTM methods. These tests and the geologic mapping program described in Attachment C2.5 verified that the subgrade consisted of the Illinoian till of the unaltered Glasford Formation.

The subgrade was considered adequate when the average in situ dry density exceeded a conservative value of 130 pcf for cohesive materials, or a minimum of 85% relative density (ASTM D-2049) was achieved at all test locations consisting of granular material. In areas where these requirements were not achieved, additional compaction and/or testing was performed to confirm the adequacy of the subgrade. Additional excavation was performed where subgrade conditions could not be improved.

Some localized pockets of sand that could not be compacted to meet the requirements were removed and dental work was performed using a flyash backfill mixture. Figure 2.5-375 illustrates the areas that received the flyash backfill.

Flyash backfill is a mixture of cement, flyash, sand, and water mixed in a central concrete batching plant. The backfill was transported and placed so as not to permit sedimentation. The material was tested in place and was considered to be acceptable when it had a strength which would yield a deflection of less than 0.25 inches under an applied load of 50 psi. A total of 90 in-place strength tests were performed on the bash placed as structural fill beneath the main power block. The minimum load used was approximately 55 psi and the maximum deflection recorded was 0.194 inches. Therefore, the bash placed beneath the power block is acceptable. The flyash mixture is described in Subsection 2.5.4.14.

2.5.4.5.1.5 Structural Fill and Backfill

Fill placement commenced following subgrade approval. Controlled compacted granular fill, Type B material, was placed to bring the base elevation of the excavation up to the grade elevations for the foundations of the various plant structures.

The dimensions and elevations of the foundation mats of the various structures, the type and thickness of the bearing strata, and the nature of the in situ soil underlying the bearing strata are summarized in Table 2.5-47. The details of the station structure foundations and the bearing strata are also shown in Figure 2.5-372.

The soil exploration program for the Type B material involved the location of a borrow area from which the structural fill and backfill could be obtained. Aerial photographic interpretation, field reconnaissance, soil exploration data, and published literature had been used in the borrow area survey.

The original proposed borrow area shown in Figure 2.5-352 was investigated by the G-series borings illustrated in Figures 2.5-152 through 2.5-161. This area, discussed in the PSAR, was not used as a source of the structural fill and backfill for the main plant. Instead, another borrow area, shown in Figure 2.5-353, was considered to be more accessible and to have material more suitable for use as Type B structural fill. However, the initial testing performed on the samples taken from the proposed borrow area is still considered valid since the materials in the two borrow areas are of similar origin.

The granular material that was used as Type B structural fill and backfill was obtained from the area designated in Figure 2.5-353. This area was investigated by drilling eleven auger Borings K-1 through K-8, K-11, K-12, and K-15, as shown in Figure 2.5-353. The logs of the borings are shown in Figures 2.5-243 through 2.5-253. The borrow area was located in alluvial deposits approximately 2.25 miles south of the station site. The fill was overlain by about 5 to 10 feet of clayey sand and clayey silt. The borrow material was a brown, poorly to well graded, fine to coarse sand with little silt, a trace of fine gravel, with coarse gravel and cobbles occasionally noted (SP, SW). A statistical average gradation of the Type B fill material is presented in Figure 2.5-351.

After the overburden was stripped from the borrow area, a dragline operation was established to excavate and stockpile the material at the borrow area. Front end loaders were used to load the material into bottom dump and end dump trucks which hauled the material to the designated stockpile area adjacent to the main plant excavation. The intermediate stockpiling and handling aided in the mixing of the borrow material to produce a homogeneous mixture.

Approximately one-half million cubic yards of this material were placed as structural fill and backfill in the plant excavation.

Laboratory testing was performed on representative bulk samples of material used as Type B structural fill and backfill. The testing included: grain-size distribution, relative density, and dynamic triaxial tests. The testing is discussed in Subsection 2.5.4.2.6.

Type B material used as structural fill to support foundation loads was placed in near-horizontal lifts. Each lift was compacted by a smooth wheel vibratory roller. The relative density was determined by the ASTM D-2049-69 test method.

A statistical analysis of the in-place density tests for the Category I structural fill beneath the plant was performed to verify the compaction requirements. The average dry density and the average relative density of the fill were determined using the 4789 nuclear and sand cone tests performed on the granular fill. These averages were computed for each one-foot interval and for the total fill.

The average relative densities for each one-foot interval range from 91.9% to 106.0%. The average dry densities varied from 129.5 lbs/ft^3 to 133.2 lbs/ft^3. The average relative density for the entire fill is 97.0%, with an average dry density of 131.5 lbs/ft^3.

Figure 2.5-428 shows the distribution of the relative density for the in-place tests. Figure 2.5-429 shows the distribution of the dry density for the in-place tests. Figure 2.5-430 shows the distribution of the moisture content of the in-place tests.

Of the 4789 tests, 175, or 3.7%, had relative densities less than the specified 85%. The locations of these 175 tests having lower densities are not concentrated at any one area or at any one elevation as shown in Figure 2.5-431. Figure 2.5-432 illustrates the grid pattern used to divide the power block fill area. A review of the areas surrounding these randomly scattered pockets of lower density material indicates that the preceeding and subsequent layers of fill were adequately compacted and met the specified density requirements. Also, the placement of additional fill above these areas of lower density increased the in-situ density in these areas. Therefore, these lower density tests will not adversely affect the integrity of the fill. Based on this summary, the structural fill beneath the main plant structures at CPS is adequate to support the structural loads and meet the other criteria used in their design.

A concrete mud mat was poured over the Type B structural fill to prevent rutting, erosion, and the provide a firm working area for the mat foundation. The specified compressive strength of the concrete used as the mud mat was 2,000 psi at 28 days. The actual compressive strength based on cylinder tests for all concrete used as a mud mat was significantly greater than the specified strength.

Type B material used as backfill, at places other than to support foundations loads, was placed in near-horizontal lifts. Each lift was compacted by either a smooth wheel vibratory roller or vibrating tamping plates (for areas adjacent to structures).

An analysis of the 3,479 in-place density tests taken on the granular backfill placed around the main plant was performed to summarize the data. Figure 2.5-482 shows the distribution of the dry density test results. The dry density ranged from 114.9 PCF to 147.2 PCF with an average value of 129.7 PCF. Figure 2.5-483 shows the distribution for the relative density test results. The relative density ranged from 56.1% to 134.8% with an average relative density of 94.5%.

A total of 42 of the 3,479 tests performed did not meet the acceptance criteria for relative density. This represents 1.2% of these tests. Twenty-six of the 42 tests represent an area in the southwest corner of the main plant excavation. This area, approximately 100 feet by 300 feet, represents approximately 6 feet of fill, and has backfill material above and below it that met the criteria. A review of the tests in this area shows that the average relative density is 81.9%. Of the remaining 16 tests that did not meet the acceptance criteria, five tests had additional correlation tests performed that met the acceptance criteria and two tests had additional tests performed to average the values. For these two tests, the averages were acceptable. Based on this review the tests that did not meet the acceptance criteria represent only isolated areas that will not be detrimental to the integrity of the backfill.

Type A cohesive material was used as backfill material over the granular material above approximately elevation 720 feet. Type A material was defined as having a plasticity index greater than 4.0 and no less than 45% passing the No. 200 sieve. The material was placed in near-horizontal lifts not exceeding 8 inches in loose thickness prior to compaction.

An analysis of the 1,703 in-place density tests taken on the cohesive backfill placed around the main plant was performed to summarize the data. Figure 2.5-479 shows the distribution of the dry density test results. The dry density ranged from 112.9 PCF to 139.2 PCF with an average value of 124.0 PCF. Figure 2.5-480 shows the distribution of the moisture content. The moisture content ranged from 6.7% to 15.3% with an average value of 11.2%. Figure 2.5-481 shows the distribution of the percent compaction. The percent compaction ranged from 82.2% to 104.0% with an average value of 93.0%.

A total of 14 of the 1,703 tests performed on the cohesive material did not meet the acceptance criteria. This represents less than 1% of these tests. Only two tests did not meet the moisture requirement, and their values were only 0.1% outside of the acceptable limits. The remaining 13 tests did not meet the compaction requirement. Of these, only seven tests had a percent compaction less than 90%. These tests represent isolated areas and they will not be detrimental to the integrity of the backfill.

Laboratory testing was performed on representative bulk samples of the material used as Type A backfill. The testing included: grain-size analysis, Atterberg limits, and Modified Proctor Compaction tests. This testing was discussed in Subsection 2.5.4.2.6. One bulk sample was taken for every 6000 yd^3 of material placed. A summary of the properties on the in-place Type A material is presented in Table 2.5-37. The moisture-density relationships are presented on Figure 2.5-350.

Flyash backfill was used as fill material in confined areas as shown in Figure 2.5-376 and Figure 4 of Question 241.8. The requirements for the fly ash backfill involve in-place testing with no compactive effort as previously stated in Subsection 2.5.4.5.1.4.

The placement of all structural fill and backfill was monitored under a comprehensive quality control program which required in-place testing for each lift of material placed. The frequency of testing required for Type B material was four in-place tests for every 10,000 ft^2 of material per lift. The frequency of testing required for the Type A material was two in-place tests for every 10,000 ft^2 of material per lift.

The construction of Unit 2 is cancelled. The majority of the excavation remains open, however, an administration building has been constructed at the north end of the excavation. Backfill behind the walls of Unit 1 has been extended and graded to drain water away from the open excavation. Erosion of the backfill due to incidental water will be prevented by the installation of a revetment composed of either gabions, cribbing, and/or a grout intrusion blanket similar to Fabriform. Water that collects within the excavation will be drained by gravity through the Unit 2 circulating water pipe to a juncture with a drain pipe that exits directly to the lake south of the screen house (Q&R 241.18).

Type B granular material from a borrow source (Borrow Area K) as shown on Figure 2.5-353 by the K-series borings, was used as structural fill and backfill for plant structures. In the PSAR the proposed source for the structural fill material was a borrow source (Borrow Area G) as shown on Figure 2.5-352 by the G-series borings. A composite sample from Borings G-18, G-19, and G-20 was used to determine static and dynamic strength properties for the structural fill material. Because of the similar nature and geologic source of these two borrow areas, the strength properties for these tests were also used for the material from Borrow Area K. These properties are:

Consolidated-Undrained Triaxial Test Data - USAR Table 2.5-15

Dynamic Triaxial Compression Test Data - USAR Table 2.5-24

Resonant Column Test Data - USAR Table 2.5-28

Dynamic Triaxial Compression Tests - USAR Figures 2.5-302 to 2.5-311

Parameters for Analysis of Rock-Soil-Structure Interaction - USAR Table 2.5-48 (for compacted structural fill)

Consolidation Test - USAR Figure 2.5-338.

Both the material used for structural fill from Borrow Area K and the material tested from proposed Borrow Area G were from alluvial deposits in the Salt Creek Valley. Revised Figure 2.5-14 shows the location of the two Borrow Areas G and K. The grain size curves for the structural fill material used and the material used for the strength property tests are shown on revised Figure 2.5-351. Both of these curves show the material as a well graded ($Cu = D_{60}/D_{10} > 4$) sand with the material used as fill being slightly coarser than the material tested.

The use of test specimen from the proposed borrow area for static strength and consolidation properties should be conservative because the coarser material of the fill should have a greater angle of internal friction at corresponding relative densities. The dynamic properties should be similar because the grain size distributions are relatively close and the pore pressure response due to dynamic loading will be similar. The densities of the material were also very close. For the test material from Borrow Area G, a relative density equal to 91.0% corresponds to a dry density of 129.4 pcf where the average dry density of 129.1 pcf for the structural fill corresponds to an average relative density equals 91.0% based on the average minimum and maximum densities of the fill material (Q&R 241.1).

2.5.4.5.1.6 General Fill

Beyond the 40-foot limit of the Category I backfill in the plant excavation, granular material was placed. This material was classified as Type B1 with the same soil characteristics as Type B material except that the gradation for the No. 200 sieve allowed for a maximum of 13% passing instead of only 10%. The placement of the general fill was similar to that of the Type B backfill except that the in-place density testing requirement was relaxed to a frequency of one test per 10,000 ft^2 per lift.

Type C cohesive backfill material was also placed beyond the 40-foot limit of the Category I backfill. This backfill was a continuation of the lifts of the Type A material placed as Category I backfill described in Subsection 2.5.4.5.1.5. A description of the Type C material is presented in Subsection 2.5.4.5.2.5. The in-place density test requirement was relaxed to one test per 10,000 ft^2 per lift. The analysis of the in-place density test results for both the granular and cohesive general fill have been included in the respective backfill summaries in Subsection 2.5.4.5.1.5.

Alternative IDOT CA-6 type backfill material may be used in non-safety-related fill applications associated with the RAT and ERAT transformer.

2.5.4.5.2 Circulating Water Screen House

2.5.4.5.2.1 Site Preparation

Site preparation and earthwork for the circulating water screen house consisted of the same operations as described in Subsection 2.5.4.5.1.1.

2.5.4.5.2.2 Excavation

Figures 2.5-378 and 2.5-379 show the details of the foundation excavation for the construction of the circulating water screen house. The bottom of the foundation mat of the circulating water screen house was approximately at elevation 653 feet. At the screen house location, the top of the Illinoian till was at approximately elevation 660 feet. The base of the excavation was established in sound Illinoian till of the unaltered Glasford Formation.

The depth of excavation was approximately 65 feet and 35 feet on the east and west sides, respectively. Construction slopes of the 3:1 and 1:1 (horizontal to vertical) for the excavations in the Salt Creek alluvium of the Henry Formation and Illinoian glacial till of the unaltered Glasford Formation, respectively, were adequate to ensure the stability of the slopes. Construction slopes of 2.0-1.5:1 (horizontal to vertical) in the Wisconsinan till of the Wedron Formation and interglacial zone materials of the weathered Glasford Formation were adequate to ensure the stability of the slopes in these materials.

2.5.4.5.2.3 Dewatering

Dewatering of the screen house was accomplished by providing a series of open drainage ditches at the outer limits of the excavation that collected the seepage water at sumps located in two corners of the excavation. The water was then pumped out as it became necessary to allow earthwork operations to be performed under dry conditions. The ditches were later backfilled with cohesive material.

2.5.4.5.2.4 Excavation Base Treatment

Pockets of loose sand exposed in the bottom of the excavation were removed to establish the foundation on sound Illinoian till of the unaltered Glasford Formation.

Subgrade verification was performed as part of a quality control program similar to that described in Subsection 2.5.4.5.1.4.

Subsequent to subgrade testing, a concrete mud mat, 6 inches in thickness, was placed over the Illinoian till to protect the subgrade and to provide a dry working surface for the foundation work.

2.5.4.5.2.5 Structural Backfill

Material placed as structural backfill around the screen house consisted of cohesive material (Type C) excavated from the main station construction area and heat sink borrow area.

Type C material was defined as having a plasticity index greater than 4.0 and no less than 45% passing the No. 200 sieve. It was placed and compacted to a minimum of 90% of the maximum dry density as determined by the ASTM D-1557 test method. The material was placed in near horizontal lifts prior to compaction. Laboratory testing was performed on representative samples used for the earthwork operations in the heat sink as well as on the material used specifically as backfill around the screen house. The moisture-density relationship for the material is shown in Figure 2.5-350. A summary of the properties of the in-place materials is the same as that presented in Table 2.5-37.

Flyash backfill was used for dental backfill work around piping and to backfill the sumps as shown in Figure 2.5-376.

A comprehensive quality control program was also followed for the backfill operations with two in-place density tests per 10,000 ft^2 per lift being required.

An analysis of the 1,742 in-place density tests taken for the cohesive backfill was performed to summarize the data. Figure 2.5-476 shows the distribution of the dry density test results. The dry density ranged from 116.3 PCF to 140.6 PCF with an average value of 125.0 PCF. Figure 2.5-477 shows the distribution of the moisture content test results. The moisture content ranged from 5.6% to 15.2% with an average value of 11.0%. The percent compaction ranged from 87.4% to 105.6% with an average percent compaction of 93.7%. Figure 2.5-478 shows the distribution of the percent compaction.

Of these 1,742 tests, 98 did not meet the acceptance criteria for either the percent compaction or moisture content. This represents 5.6% of the screen house backfill tests. However, 69 of the 98 tests represent the area where the SSWS pipeline enters the screenhouse. In this area, a 95% degree of compaction was required. The tests performed in this area were evaluated and the average percent compaction for this area was greater than 93% and was considered to be acceptable. The reduction in the compaction effort will not be detrimental to the function of this fill.

Of the remaining 29 of the 98 tests, only four had a percent compaction less than 89%. One of these tests had two other tests performed near the same location with the average percent compaction of the three tests being greater than 90%. Six of these 29 tests did not meet the moisture content acceptance criteria only. However, their moisture contents were generally within 1.5% of the acceptable range, and will not be a detriment to the fill.

In conclusion, the 98 tests that did not meet the acceptance criteria were found to represent only small areas which will not be detrimental to the integrity of the backfill.

2.5.4.5.3 SSWS Outlet Structure and Pipelines

2.5.4.5.3.1 Site Preparation

Site preparation and earthwork for the shutdown service water system (SSWS) outlet structure and pipelines consisted of the same operations as described in Subsection 2.5.4.5.1.1.

2.5.4.5.3.2 Excavation

The excavation for the SSWS outlet structure extended from the existing grade to the Illinoian till of the unaltered Glasford Formation approximately at elevation 655 feet. The excavated slopes from elevation 655 feet to 662 feet were near vertical and were approximately 5 feet from the structure itself. The slopes above elevation 662 feet were cut back on a 2:1 (horizontal to vertical) for construction purposes. The final slope configuration around the SSWS outlet structure is discussed in Subsection 2.5.5.1.2. The excavation and structural fill placed beneath the structure is illustrated in Figure 2.5-381.

Excavation was performed along the SSWS pipeline alignments between the screen house and the station site and between the outlet structure and the station site. A longitudinal subsoil profile along the SSWS pipeline is presented in Figures 2.5-486 and 2.5-487. Typical transverse sections illustrating the concrete mudmat, flyash mixture, pipe, and backfill materials are shown on Figure 2.5-488. Zones of soft and loose material were removed as indicated by overexcavation beneath the pipeline as shown on Figures 2.5-486 and 2.5-487. (Overexcavation is considered to be any excavation greater than 1.5 feet below the bottom of the lower pipe.) Minor seepage into the pipeline excavation was pumped as it became necessary. This excavation was normally dry after rain.

2.5.4.5.3.3 Dewatering

Minor seepage into the excavation was diverted around the outer limits of the outlet structure excavation by open ditches. The water was drained by gravity away from the excavation into a larger collector ditch from which the water was pumped as necessary.

2.5.4.5.3.4 Excavation Base Treatment

The base of the excavation for the SSWS outlet structure was established on sound Illinoian till. Pockets of loose material were removed prior to subgrade testing and approval.

A concrete mud mat, with a minimum thickness of 4 inches, was placed on the approved subgrade for the outlet structure to protect it from exposure.

Soft material encountered immediately behind the outlet structure was overexcavated and Type A cohesive material was placed there. A total of 5 feet of Type A material was placed immediately behind the structure to replace the excavated soft materials. An analysis of the 50 in-place density tests performed on the cohesive backfill around the outlet structure was made to summarize the data. The dry density of the material ranged from 122.9 PCF to 131.0 PCF with an average dry density of 126.5 PCF. Figure 2.5-468 illustrates the distribution of these values. The moisture content of the material ranged from 7.5% to 13.3% with an average of 11.0%. Figure 2.5-469 shows the distribution of the moisture content. The percent compaction ranged from 91.9% to 98.3% with an average value of 94.9%. Figure 2.5-470 shows the distribution of the percent compaction for these tests. Type A material is described in Subsection 2.5.4.5.1.5.

A concrete mudmat having a minimum thickness of 4 inches was placed beneath the SSWS pipeline either over the approved subgrade or structural fill along the pipeline.

2.5.4.5.3.5 Structural Fill and Backfill

Type B granular fill material, as discussed in Subsection 2.5.4.5.1.5, was placed as structural fill directly over the mudmat beneath the outlet structure from approximately elevation 655 feet to 662 feet. This material was placed in near horizontal lifts. An analysis was performed on the 15 in-place density tests performed on the Type B granular fill. The dry density of this material ranged from 127.4 PCF to 134.2 PCF with an average dry density of 131.1 PCF. Figure 2.5-466 shows the distribution of the dry density test results. The relative density, as determined by ASTM D-2049, ranged from 89.8% to 99.6% with an average relative density of 94.9%. Figure 2.5-467 shows the distribution of the relative density test results. All of these tests met the acceptance criteria of a minimum of 85% relative density. A thin concrete seal was placed over the Type B material to protect it from runoff water.

Between the elevations of 662 feet and 669 feet, fly ash mixture backfill was placed and tested as described in Subsection 2.5.4.5.1.4. A 12-inch thick apron of the fly ash mixture backfill was also placed along the two side walls of the outlet structure. Four in-place strength tests were performed on the fly ash mixture beneath the SSWS outlet structure. The maximum deflection was 0.022 inches for a load of 63.6 psi. This is less than the allowable deflection of 0.25 inches for a 50 psi load.

A total of 24 in-place tests were performed on the fly ash mixture placed along the SSWS pipeline. A load of 71.7 psi was used for all of these tests with a maximum deflection of 0.174 inches being recorded. Therefore, the tests performed for the SSWS pipeline and outlet structure are acceptable.

Flyash mixture backfill was placed around the SSWS piping as shown on Figure 2.5-488. Structural backfill was then placed and compacted over the pipes.

Type B granular material was used as fill around the lower pipes immediately adjacent to the main plant structures. A summary of the 59 in-place tests performed in this area was made to summarize the data. The dry density of this fill ranged from 121.6 PCF to 132.7 PCF with an average value of 126.6 PCF. Figure 2.5-471 shows the distribution of the dry density test results. Figure 2.5-472 shows the distribution of the relative density test results. The relative density ranged from 85.6% to 118.0% with an average value of 100.8%. All of these tests met the acceptance criteria of 85% relative density.

Cohesive material was used as fill around the SSWS pipeline in all the remaining areas. An analysis of the 524 in-place tests taken on the cohesive material was performed to summarize the data. Figure 2.5-473 shows the distribution of the dry density test results. The dry density ranged from 116.2 PCF to 133.8 PCF with an average dry density of 122.3 PCF. Figure 2.5-474 shows the distribution of the moisture content for the tests. The moisture content ranged from 6.2% to 14.2% with an average value of 11.1%. Figure 2.5-475 shows the distribution for the percent compaction. The percent compaction ranged from 89.1% to 101.0% with an average value of 94.4%. Only seven of the 524 in-place density tests did not meet the acceptance criteria for percent compaction of this fill material. One of these tests also did not meet the moisture acceptance criteria. These failing tests represent 1.3% of the tests performed for the pipeline. As previously stated, the lowest percent compaction recorded was 89.1%. Also, these seven failing tests represent only isolated areas along with pipeline. Therefore, the material represented by these tests will not be detrimental to the integrity of the pipeline fill.

Section C-C on Figure 2.5-488 illustrates the use of the flyash mixture as it was placed within 15 feet of the bends in the SSWS pipeline. The flyash mixture was used as bedding and placed vertically up to 1/6 of the diameter of the pipe. Styrofoam, 6 inches in thickness, was placed between the flyash mixture bedding to made the bedding for each pipe independent of each other. Structural cohesive fill and backfill was then placed and compacted as previously discussed.

Information follows for the buried shutdown service water system (SSWS) piping outdoors. There is no ECCS piping buried outdoors. (Q&R 241.8)

(a) A longitudinal subsoil profile along the SSWS pipeline is presented in Figures 2.5-486 and 2.5-487. The excavation line for the pipeline is illustrated. The zones of soft and loose material that were removed are shown as over-excavation beneath the pipeline. (Overexcavation is considered to be any excavation greater than 1.5 feet below the bottom of the lower pipe.)

(b) Transverse cross sections showing the pipe, concrete mudmat, and all backfill materials are presented in Figure 2.5-488.

(c) The details of the backfill placement near the connection between pipes and structures are shown in Section F-F of Figure 2.5-488. The estimated total settlement for the structure (Diesel Generator Building) where the SSWS pipes enter is 1 inch. This settlement is based on the estimated total settlement of the structure (Figure 2.5-436) beginning January 1, 1979. This is the approximate date of the connection of the first pipe to the structure. If it is assumed that the pipeline will not settle, the estimated differential settlement between the pipeline and structure will also be approximately 1 inch.

(d) A figure showing the fly ash placement from the SSWS outlet structure and along the pipeline to the main plant is shown in Figure 2.5-496. (Q&R 241.8).

2.5.4.6 Groundwater Conditions

A discussion of the history of the groundwater conditions, monitoring of piezometers, and groundwater conditions used in analyses is presented in Subsection 2.4.13.

A discussion of the control of groundwater and seepage in the open excavations is presented in Subsection 2.5.4.5.1.3, 2.5.4.5.2.3, and 2.5.4.5.3.3 for the main plant, screen house, and outlet structure, respectively.

2.5.4.7 Response of Soil and Rock to Dynamic Loading

The parameters utilized on soil-rock-structure interaction analyses are presented in Table 2.5-48. The static soil properties presented in this table were based on evaluation on laboratory consolidation and triaxial test data. The strain dependent dynamic moduli and damping values were evaluated on the basis of geophysical results and laboratory dynamic triaxial and resonant column tests. The selected design parameters reflect both the results of the tests performed during the PSAR investigation and properties previously developed for similar soils.

The present design of walls and slabs enveloped the conditions of Unit 2 being presented and Unit 2 structures being deferred.

In developing the soil-spring constants for soil-structure interaction being performed in response to Question 220.26, the overburden and confining pressure under the Unit 1 mat have been used. However, under the present condition where only one unit is constructed, the confining pressure under the area of Unit 2 excavation is smaller. This smaller confining pressure will lead to lower shear modulus values for the 20-foot-thick structural fill layer. A sensitivity study performed on soil properties in answer to Question 220.15 shows that lower modulus values lead to lower structural responses (Q&R 241.19).

2.5.4.8 Liquefaction Potential

2.5.4.8.1 Structural Fill

The liquefaction potential of the structural fill beneath the structure was evaluated on the basis of the simplified procedure described by Seed and Idriss (Reference 85). The procedure is based on both theoretical considerations and descriptions of site conditions where liquefaction was known to have occurred or not to have occurred under earthquakes of known or estimated magnitudes. The liquefaction potential of a granular soil deposit is related to:

a. the grain-size characteristics of the sands,

b. the relative density,

c. the position of the groundwater table,

d. the intensity and duration of ground shaking, and

e. the number of significant stress cycles produced by the earthquake.

Results of the evaluation are presented in Table 2.5-49 for various depths in the structural fill beneath the structure. Evaluation is based on the safe shutdown earthquake postulated for the site (Subsection 2.5.2.6). The number of significant stress cycles is anticipated to be five cycles according to a correlation between the significant stress cycles and magnitude of earthquake established by Seed et al. (Reference 85). However, in the analysis ten significant cycles were conservatively used for the safe shutdown earthquake. The structural granular fill Type B material was compacted to a minimum relative density of at least 85% determined by ASTM D-2049 method of compaction.

The maximum shear stresses were computed assuming that the soil behaves as a rigid body. The rigid body stresses were corrected using a stress reduction coefficient to account for the fact that the soil actually behaves as a deformable body. The average equivalent uniform shear stress during the earthquake is estimated to be 65% of the computed maximum shear stress. The stresses required to produce liquefaction in ten cycles were computed using the stress ratio (ratio of one-half of the cyclic vertical stress to confining pressure) obtained from laboratory liquefaction tests. The laboratory liquefaction test results are shown in Figure 2.5-349. Because of the difference between the field and laboratory stress conditions and the limitations in testing equipment and procedures, a correction factor of 0.70 was applied to the laboratory values to obtain the cyclic shear stresses required to produce liquefaction in the field (Reference 86).

The liquefaction potential of the structural fill was evaluated by computing the factor of safety with respect to liquefaction at various depths during the safe shutdown earthquake. The factor of safety is defined as the ratio of the cyclic shear stress required to produce liquefaction to the average uniform cyclic shear stress induced by the earthquake. The calculations are based on ten significant stress cycles. The water table was assumed to be located at elevation 730 feet, which is 6 feet below grade elevation.

The results of the evaluation shown in Table 2.5-49 indicate that the factor of safety with respect to initial liquefaction during the safe shutdown earthquake is 2.03 for the structural fill. Therefore, there is no liquefaction potential for the structural fill beneath the structure which is to be compacted to a minimum relative density of least 85% as obtained by ASTM D-2049 method of compaction.

Question 241.9 concerning the liquefaction potential of the natural material in the vicinity of ECCS piping is assumed to apply to buried shutdown service water system (SSWS) piping outdoors. There is no ECCS piping buried outdoors.

The material used as structural backfill around the SSWS piping consisted of concrete, bash, and Type A cohesive material. These materials are not susceptible to liquefaction and thus not likely to liquefy.

As discussed in Attachment C2.5, Geologic Mapping, the subgrade for the SSWS piping consisted of the Wisconsinan Till of the Wedron Formation. This till consists of cohesive material with isolated and discontinuous pockets of sand and silt randomly distributed within the till. The cohesive material is not susceptible to liquefaction and thus not likely to liquefy.

Some small isolated sand pockets were encountered during the excavation for the SSWS pipeline. These are shown on Figure C2.5-23. In-place density tests were performed in these areas. The results of these tests indicate that these sand pockets have an in-place relative density greater than 82.5%. Based on the facts that the sand pockets are confined, discontinuous, and that they have an in-situ relative density greater than 82.5%, they are not considered to be very likely to liquefy (Q&R 241.9).

The minimum factors of safety for sand elements were obtained by comparing the shear stresses required to cause single-amplitude shear strain of 5% (see Figure 2.5-413) with the equivalent shear stresses induced by the earthquake. It can be seen from Table 2.5-68 that the 5% strain occurs before the initial liquefaction starts. Therefore, the criterion of 5% strain is conservative, and it assures that the sand elements will not liquefy during the earthquake (Q&R 241.16).

2.5.4.8.1.1 Structural Fill Subjected to New Madrid Type Earthquake

The liquefaction potential of the structural fill was analyzed in the same way for the New Madrid type earthquake (Subsection 2.5.2.6) as for the safe shutdown earthquake. The details of the analyses are the same in both cases except for the earthquake parameters. The changes in the parameters for the analyses of the New Madrid type earthquake are:

a. a maximum ground acceleration 0.13g, and

b. the number of significant stress cycles was assumed to be 30.

The results of the evaluation shown in Table 2.5-50 indicate that the minimum factor of safety with respect to initial liquefaction during the New Madrid type earthquake is 2.14 for the structural fill.

2.5.4.8.2 Sand Lenses

The soils under the station site above elevation 683 feet were removed except in a few isolated areas where sound Illinoian till of the unaltered Glasford Formation was encountered at approximately elevation 686. The excavation was then filled with Type B granular material compacted to a minimum relative density of 85% as determined by the ASTM D-2049 test method and described in Subsection 2.5.4.5.1.5. The sand lenses below elevation 680 feet were examined for liquefaction potential by evaluating the soil borings under the station mat including the seven additional soil borings, P-33A and P-50 through P-55, drilled specifically for this purpose. Further discussion of the borings within the station mat is given in Subsections B2.5.2 and B2.5.2.1.

Based on the physical data indicated on the boring logs, which includes standard penetration test values, relative density, grain size analyses, confining pressures, material type, and in situ dry density, it is concluded that the sand lenses are not susceptible to liquefaction. Therefore, no settlements due to liquefaction are anticipated. The facts leading to this conclusion are discussed in detail in Attachment B2.5.

2.5.4.9 Earthquake Design Basis

The response spectra defined in Subsections 2.5.2.6 and 2.5.2.7, and presented in Figures 2.5-296 and 2.5-297, specify the earthquakes which are used in the design and analysis.

2.5.4.10 Static Stability

2.5.4.10.1 Foundation Exploration and Testing Program

Geologic, seismologic, and foundation conditions of the site were evaluated by the boring program presented in Subsection 2.5.4.3. Subsection 2.5.4.2 describes in detail the static and dynamic engineering properties of the materials underlying the site.

Figure 2.5-271 shows the location of the borings on the site. Borings used in the analysis of station foundation conditions are listed in the following tabulation:

BUILDING	NO. OF BORINGS	BORING NUMBER
Fuel, containment, and auxiliary buildings (Unit 1)	5	P-14, P-37, P-38 P-54, P-55
Fuel containment, and auxiliary buildings (Unit 2)	3	P-18, P-29, P-30
Turbine building (Unit 1)	5	P-39, P-40, P-41, P-52, P-53
Turbine building (Unit 2)	7	P-15, P-31, P-32, P-33, P-33A, P-50, P-51
Radwaste	2	P-10, P-36
Control	1	P-35
Immediately adjacent to station area	7	P-7, P-9, P-11, P-34, P-42, P-43, P-47

Engineering analysis has shown that the supporting capacity of the Illinoian till and underlying lacustrine and pre-Illinoian soils is far in excess of the pressures imposed by the mat foundations under static and dynamic loading conditions.

Settlement analysis indicates that the anticipated foundation loads imposed on the Wisconsinan till and the interglacial zone materials which overlie the Illinoian glacial till would cause excessive settlement of the station complex. These soils were removed and replaced with a controlled, compacted, granular Type B fill as discussed in Subsection 2.5.4.5. The compacted fill bears directly on the underlying hard Illinoian glacial till strata. The hard Illinoian glacial till consists of gray to brown clayey silt with occasional gravel-sized particles. The underlying lacustrine and pre-Illinoian soils are hard and consist of clayey and sandy silts with occasional gravel-sized particles. The underlying lacustrine and pre-Illinoian soils also exhibit high density and high shear strength characteristics. The approximate thickness of the Illinoian till is 120 feet and the combined thickness of the underlying lacustrine and pre-Illinoian soils is approximately 60 feet. These soils are immediately underlain by thinly bedded Pennsylvanian limestone and shale.

2.5.4.10.2 Bearing Capacities

The fill beneath the station foundations was constructed using selected granular materials (see Subsection 2.5.4.5.1.5) placed using controlled compaction procedures. The compacted fill has excellent supporting capacity to transmit the anticipated structural loads to the underlying hard Illinoian glacial till without excessive foundation settlements.

Ultimate bearing capacities and factors of safety for the station mat foundations, the circulating water screen house, and the ultimate heat sink outlet structure are presented in Table 2.5-63. These factors of safety were calculated by conventional bearing capacity analyses assuming a local shear failure condition. It was assumed that the subsoil beneath the foundations is uniform and the mats under the various components of the station are structurally independent with respect to foundation loading and support. The total static foundation loads refer to total dead loads and equipment operating loads.

Since the rigidity of the soil will increase during dynamic loading which is usually associated with smaller strain, the ultimate bearing capacities of the underlying cohesive soils will be larger under dynamic loading than for static conditions; and as a result, the factors of safety under seismic loading conditions would not be substantially reduced.

The interaction of the subgrade and the mat foundations was investigated to evaluate the effects of slab deflection under static loading conditions. A static modulus of subgrade reaction of 25 to 300 psi was utilized in the analysis of the mat foundations.

The CPS power block complex is supported on a monolithic basemat. For combined SSE and pool dynamic loads (SRV and LOCA), the maximum foundation-bearing pressure and displacement under the mat can be found in calculation SDQ12-21DG06.

2.5.4.10.3 Settlement

The results of consolidation tests on soil samples obtained from various elevations at the station site indicate that the Wisconsinan glacial till and the interglacial zone materials were more compressible than the underlying Illinoian glacial till. Consequently, mat foundations established in the near-surface solid would undergo excessive settlement when subject to moderately high foundation pressures. These materials were, therefore, considered unsuitable as bearing strata for the mat foundations, and were removed by excavating to the top of the Illinoian glacial till. Type B controlled compacted granular fill was placed over the subgrade to bring the bottom of the excavation up to foundation grade.

The results of consolidation test data are presented in Figures 2.5-320 through 2.5-346 and summarized in Table 2.5-62. Consolidation tests were performed on representative samples as described in Subsection 2.5.4.2.3.2. The test results indicate that the Illinoian and pre-Illinoian tills are overconsolidated. Estimates for the maximum past consolidation stress based on Casagrande method of construction (Reference 87) indicate that the preconsolidation pressure of the Illinoian glacial till lies between 14.0 and 25.0 ksf (Table 2.5-62). Also, vertical stresses in the subsoil due to structural loadings do not exceed the difference between the maximum past preconsolidation pressure and the existing effective overburden pressure. Hence, values of the recompression index, C_r, were used to estimate the settlement.

The soils were divided into eight strata in the settlement analysis. The recompression indices assigned to each soil stratum are presented in Table 2.5-66. For computing the rate of settlement, an average value of coefficient of consolidation of 11.4 sq. ft. per day is used for the entire soil profile.

Settlement analyses have been performed to compute the settlement of plant structures according to the construction sequence. Excavation for the plant structures began in the fall of 1975 and was completed in about nine months. Backfill of the controlled compacted fill and construction of Unit 1 then followed. For analysis purposes, a three-month period for the placement of controlled compacted fill and a four-year period for the construction of Unit 1 structures are used. It is further approximated that the construction period of the mat foundation of Unit 1 is one year. The construction of Unit 2 has been cancelled. The final settlement of Unit 1 was less than that shown in Figure 2.5-433.

The grade elevation of the station site is approximately 736 feet. The design groundwater level is assumed to be elevation 730 feet (Subsection 2.4.13). The foundation elevations and static loads for each of the plant structures are shown in Table 2.5-63.

The construction time sequences are divided into excavation, placement of the controlled compacted fill, and construction of plant structures. There is no settlement due to the dewatering process employed during construction (Section 2.5.4.5.1.3). The settlement due to applied building loads is computed based on the gross foundation loads minus the uplift pressures.

The SETTLE computer program described in Appendix C is used to compute settlements. SETTLE uses the comparison index method to perform the settlement analysis. The stress increment due to applied external loads is computed using Bouissinesq's formula.

The settlement due to the applied building loads is computed by assuming that the structural foundation system is either completely rigid or completely flexible. The settlement is taken as the average of the results obtained from these two cases. For the completely flexible case, the rigidity of the foundation and superstructure system is neglected. The effective foundation pressure is applied at the foundation level directly on top of the soil. For the completely rigid foundation case, the distribution of contact pressure due to foundation rigidity is taken into account by considering linear settlement. An iterative procedure is used to make the settlement pattern of the foundation and the subsoil compatible. This iterative procedure is included in the SETTLE program.

The settlement time histories for the main plant are computed using the one-dimensional consolidation theory. Each loading (i.e., excavation, structural fill, applied building loads) is applied independently and the resulting settlement time history is obtained by superposition. The initiation of settlement of the plant structures is taken at the completion date of the construction of the Unit 1 mat foundation. This point on the computed settlement time history curve, therefore, marks the origin of the datum line for defining plant settlement. The computed time histories of plant settlement at four settlement points are shown in Figures 2.5-434 through 2.5-437. The contours of the computed final settlement for the main plant area are shown in Figure 2.5-433. The differential settlements between adjacent buildings are not presented because the entire power block is on one monolithic mat foundation and various buildings are also rigidly connected with continuous shear walls and diaphrams.

An instrumentation system of settlement monuments was established at various locations throughout the main plant buildings to monitor the settlement. This system is discussed in Subsection 2.5.4.13. The locations of those settlement monuments are shown in Figure 2.5-382. The graphical plots of measurement readings recorded to date (January 1984) for those settlement monuments are presented in Figure 2.5-438. A comparison between the calculated final settlement and the measured settlements to date (January 1984) for all the settlement monuments is presented in Table 2.5-67. The calculated and measured settlement time history values are compared for settlement monuments C4 (Containment Building, Unit 1), T2 (Turbine Building, Unit 1), D3 (Diesel Generator Building), and R3 (Radwaste and Off-Gas Buildings), as shown in Figures 2.5-434 through 2.5-437 respectively. It should be noted that the first measurement readings are established on the theoretical settlement curves, and subsequent measurements are plotted with reference to those initial values. The comparisons show that the calculated settlements agree reasonably well with the measured values.

2.5.4.10.4 Lateral Earth Pressures

Based on anticipated groundwater elevations described in Subsection 2.5.4.6, all substructures were designed to resist full hydrostatic groundwater pressure at all levels below elevation 730 feet. All mat foundations established below elevation 730 feet were designed to resist hydrostatic uplift pressures.

Subsurface walls were designed to resist lateral pressures induced by soil and groundwater under both static and dynamic loading conditions.

The analysis for lateral stability of the power station, under static and dynamic loading conditions was performed for the following situation:

Only Unit 1 constructed with an open excavation for Unit 2.

Adequate factors of safety for lateral stability exist for the above condition (Q&R 241.20).

The values of the lateral earth pressures due to the soil, water, surcharge, and dynamic loads were determined using the following equations:

SOIL

$$Ps = K \gamma_w h, \text{ for } h < h1 \qquad (2.5\text{-}5)$$

$$Ps = K \gamma_w h1 + K(\gamma_S - \gamma_H)(h - h1), \text{ for } h > h1$$

WATER

$$Pw = 0, \text{ for } h < h1 \qquad (2.5\text{-}6)$$

$Pw = \gamma_H (h - h1)$, for $h > h1$

SURCHARGE

$Pc = KS$ (2.5-6)

DYNAMIC INCREMENT

$Pe = K_h (H_o - h) \gamma_w$ for $h < h1$ (2.5-8)

$Pe = K_h (H_o - h) \gamma_w$ for $h > h1$ (2.5-9)

where:

H	=	height of wall (ft)
h	=	distance from grade to the point where pressure is to be determined (ft)
h1	=	distance from final grade to the groundwater table (ft)
K	=	horizontal earthquake acceleration/gravity (.10 OBE, .25 SSE)
Pc	=	lateral pressure due to surcharge (ksf)
Pe	=	lateral pressure due to earthquake at a distance h below grade (ksf)
Ps	=	lateral pressure due to soil at a distance h below grade (ksf)
Pw	=	lateral pressure due to water at a distance h below grade (ksf)
S	=	surcharge loading, .5 ksf for construction, 1.0 ksf for E-70, .3 ksf for H-20
γ_H	=	density of water (.0624 kcf)
γ_s	=	density of saturated soil (.1374 kcf)
γ_w	=	density of wet soil (.132 kcf)

(a) The static coefficient of lateral earth pressure of 0.47 was calculated using an angle of internal friction of 32°. This is equal to the at-rest earth pressure coefficient for cohesive (clayey silt and silty clay till) backfill used at CPS. Granular backfill has also been used for backfill around Category I structure walls. The at-rest earth pressure coefficient for granular backfill was calculated using an angle of internal friction of 38°. The static coefficient of lateral earth pressure for the granular backfill is 0.38 which is less than the value (0.47) used and therefore is conservative.

(b) The dynamic water pressure is considered in combination with dynamic soil pressure and is accounted for by using density of saturated soil in Equation 2.5-9, Subsection 2.5.4.10.4.

(c) The lateral earth pressures due to soil, water, and dynamic loads are considered in the design by using equations given in Subsection 2.5.4.10.4. The plot showing these lateral earth pressures is shown in Figure 2.5-492. Soil-Structure Interaction Analysis as described in Subsection 3.7.2 does not give lateral pressures. If the model synthesis approach used for the analysis, only the modal properties (i.e., frequencies, mode shapes, modal damping values, and participation factors) are extracted from the soil model. Any other responses since they are not of interest for the interaction analysis (Q&R 241.7).

2.5.4.11 Design Criteria

The design criteria used in the design of Seismic Category I structures are discussed in the following subsections:

a. liquefaction potential, Subsection 2.5.4.8;

b. bearing capacity, Subsection 2.5.4.10.2;

c. settlement, Subsection 2.5.4.10.3;

d. static slope stability, Subsection 2.5.5.2.3; and

e. dynamic slope stability, Subsection 2.5.5.2.4.

2.5.4.12 Techniques to Improve Subsurface Conditions

Localized areas and pockets of loose granular materials were encountered in the base of the excavations for some of the Category I structures. These materials were either compacted and tested or removed. Depressions created by these minor excavations were filled with a flyash backfill mixture. These operations are discussed in Subsections 2.5.4.5.1.4, 2.5.4.5.2.4, and 2.5.4.5.3.4.

2.5.4.13 Subsurface Instrumentation

An instrumentation system of settlement points was established at various locations throughout the buildings at the main plant site. Figure 2.5-382 shows the location of these devices.

Each settlement point consists of a brass monument imbedded in the rough concrete. Reference points were located around the structures from which easy accessibility could be made.

Most settlement points were measured at a minimum frequency of once every four calendar months until the plant settlement was considered stabilized. The graphical plots of measurement readings recorded versus the time for all settlement points are shown in Figure 2.5-438.

2.5.4.14 Construction Notes

Instead of using either cohesive or granular backfill in small, confined areas, a flyash mixture was used. The flyash backfill was a mixture of cement, flyash, sand, and water mixed to the following approximately proportions determined by weight per cubic yard: cement - 50 to 100 lb;

flyash - 450 lb; sand - 2800 lb; and, water - 425 lb. The material was mixed in a central concrete batching plant, transported by mixer trucks, and placed in the designated areas so as not to permit sedimentation. The material was tested in place following a curing time long enough to give the material a strength which yields a maximum deflection of 0.25 inches under an applied pressure of 50 psi. A total of 237 in-place strength tests were performed for the acceptance of the flyash mixture backfill placed beneath and around the structures including the main power block, screenhouse, outlet structure, and the pipelines. The minimum load used for the test was approximately 55 psi. The maximum deflection recorded was 0.194 inches. Therefore, the tests performed on the bash were acceptable.

The use of the flyash mixture backfill was restricted to areas beneath and around piping and for dental work on the subgrade and in confined areas. The degree of compaction of either granular or cohesive materials in these areas by conventional means could not be guaranteed due to the size awkwardness of the compaction equipment. Also, the ease of placement as well as the lack of any compaction requirements for the flyash backfill made its use more practical in these small and confined areas.

2.5.4.14.1 Main Plant

The base of the excavation for the main plant was overexcavated in several areas because the inplace density of the subgrade did not meet the requirements as stated in Subsection 2.5.4.5.1.4. Depressions created in these areas were backfilled with a flyash mixture as described in Subsection 2.5.4.5.1.4.

Structural backfill for the main plant structures consisted of Type B and B1 granular material. This material was utilized along with the Type A and C cohesive materials. Also, the flyash backfill was used around some piping in the main plant. These backfill operations are discussed in Subsections 2.5.4.5.1.5 and 2.5.4.5.1.6.

The proposed borrow area for the Type B structural fill for the main plant was not used as the source of the granular fill. A new borrow area was developed as discussed in Subsection 2.5.4.5.1.5.

2.5.4.14.2 Circulating Water Screen House

A flyash backfill mixture was used as backfill material around piping and for dental work in filling in sumps. This is discussed in Subsection 2.5.4.5.2.5.

2.5.4.14.3 SSWS Outlet Structure and Pipelines

The subgrade beneath the SSWS outlet structures was overexcavated due to the presence of soft material at the proposed foundation elevation. Type B granular material was used as structural fill beneath the structure to attain the design foundation elevation. This is discussed in Subsections 2.5.4.5.3.2 and 2.5.4.5.3.5.

A flyash mixture was used as fill material around the SSWS outlet structure as well as around the SSWS pipelines. This is discussed in Subsection 2.5.4.5.3.5.

2.5.4.14.4 Main Plant Excavation

The excavation for Unit 2, adjacent to the plant east side of Unit 1, will remain open as shown on Figure 2.5-484A. An administration building has been constructed in the Unit 2 excavation, as shown on Figure 2.5-484B. The slopes of the backfill adjacent to the Unit 1 structures will be graded as illustrated by Sections G-G and H-H. After grading, a revetment composed of a grout intrusion blanket will be placed on these slopes to protect them against erosion due to runoff. A berm, consisting of compacted Type A cohesive material or un-reinforced concrete will be placed around the perimeter of the excavation to divert flood water runoff from entering the excavation. No berm will be placed across the construction ramp.

Figure 2.5-484A also illustrates the drainage system that is utilized to drain any precipitation that enters the excavation. As shown on Section A, a flap gate will be used to prevent a backflow from the lake into the excavation if the lake level rises above the invert elevation of the drainage system. In addition to the drainage system, all openings in the Unit 1 building below grade level that are exposed in the Unit 2 excavation will be closed and waterproofed.

2.5.5 Stability of Slopes

2.5.5.1 Slope Characteristics

The alignment of the cooling lake main dam is shown in Figure 2.4-13 and a typical cross section showing the as-built details is presented in Figure 2.4-14. The layout of the ultimate heat sink and the as-built features of the submerged earthfill are shown in Figures 2.5-384, 2.5-385, 2.5-386, and 2.4-24. There are no natural or manmade slopes in the immediate vicinity of the plant whose failure would adversely affect the safety of the power plant.

2.5.5.1.1 Main Dam

The main dam was constructed with side slopes of 3:1 (horizontal to vertical) for both the upstream and downstream sides. The dam section is a homogeneous embankment constructed using the silty clay materials of the Wisconsinan glacial till of the Wedron Formation (Type A) obtained from the borrow areas. A downstream drainage blanket and a ditch were provided to drain away the small seepage that will occur through the dam. The drainage blanket will also help lower the phreatic line and prevent seepage from emerging on the downstream slope of the dam, thus preventing the softening and erosion of the downstream slopes. A typical cross section is illustrated in Figure 2.4-14.

The upstream slope of the dam is protected against destructive wave action by means of rock riprap and bedding material. The riprap design and other slope protection for the main dam is discussed in Subsections 2.4.8 and 2.5.6.4.2.2.

The average soil properties, based on laboratory tests performed on the borrow materials, are summarized in Table 2.5-52. The subsurface exploration and local geologic features at the main dam site are discussed in Subsections 2.5.4.3, 2.5.4.4, and 2.5.6.2.2.

2.5.5.1.2 Ultimate Heat Sink

The subsurface soil, rock, and groundwater conditions at the ultimate heat sink were explored by drilling 60 test borings (H-borings) at the locations indicated in Figure 2.5-16. Twenty-seven borings were drilled for the submerged dam, its abutments, and vicinity; twenty-three, for the excavated and natural slopes; six, for the baffle dike and its abutment; and four, for the screen house and outlet structures.

The static and dynamic properties of the foundation soils and the materials used in the construction of the submerged dam were evaluated through a comprehensive drilling and testing program described in Subsections 2.5.4.2 and 2.5.5.3.2. This program also included evaluation of the properties of the soils that form the slopes of the cut areas of the heat sink.

The as-built features of the ultimate heat sink submerged dam and baffle dike are shown in cross sections depicted in Figures 2.5-384 and 2.4-24.

The submerged dam and the baffle dike were constructed using the Wisconsinan Glacial Till of the Wedron Formation (Type A material) removed from borrow areas located south of the heat sink and shown in Figure 2.5-384. Side slopes of 5:1 (horizontal to vertical) for both the upstream and downstream sides of the submerged dam and the baffle dike were constructed and were determined to be stable under static and seismic loading conditions.

Slope protection for the baffle dike, the submerged dam, and its abutments was provided by a soil-cement mixture placed as shown in Figures 2.5-386 and 2.4-24. The mixing and placement procedures for the soil-cement are described in Subsection 2.5.6.4.1.2.

The stability of the slopes bordering the ultimate heat sink was examined by preparing ground profiles based on the original topography and modified as necessary by the aerial survey taken prior to the closing of the submerged dam and lake filling (Figure 2.5-385). Soil profiles based on the boring logs and field conditions used in the analysis are provided in Figures 2.5-387 through 2.5-392.

The maximum final slope of the natural ground on the south and east sides of the heat sink below elevation 690 is 5:1 (horizontal to vertical). Final slopes for the compacted fill material for the baffle dike abutment on the east side above elevation 675 is 4:1 (horizontal to vertical). The final excavated slopes in the vicinity of the screen house and outlet structure on the east side are 5:1 (horizontal to vertical) below elevation 690 and 3.5:1 (horizontal to vertical) above elevation 690. The remaining final slopes on the east side are flatter than 3.5:1 (horizontal to vertical) above elevation 690. The natural slopes above elevation 690 on the south side are flatter than 5:1 (horizontal to vertical), except at the locations of Section Y'-Y' and G-G, as shown in Figure 2.5-385. The maximum slope of the northern natural ground above elevation 668.5 is 7:1 (horizontal to vertical). The submerged dam provides the boundary along the western section of the heat sink.

The subsurface conditions in the ultimate heat sink area and adjacent to the valley walls are described in Subsections 2.5.5.2.3.2 and 2.5.6.4.1. The soil properties and other conditions considered in the stability analyses are presented in Subsection 2.5.5.2.3. The dynamic stability analysis is described in Subsection 2.5.5.2.4. Cyclic triaxial tests have been performed for determining the dynamic properties and the results are discussed in Subsection 2.5.5.2.4.

At the west side of the ultimate heat sink, where the submerged dam was constructed, the ground surface elevation ranges from 670 to 673 feet in the floodplain, as revealed by the elevations of Borings H-36, H-35, H-7, H-5, H-4, and H-34. Boring H-37 (693.9 feet elevation) is located at the south abutment on the south valley wall; Boring H-51 (690.3 feet elevation) approximately at the location of the north abutment on the north side. The locations of these borings can be found in Figure 2.5-16.

The subgrade preparation for the submerged dam removed all unsuitable soils revealed by the borings. The embankment was constructed on the Illinoian glacial till of the unaltered Glasford Formation.

The construction of the embankment required the excavation of the soils at the abutments of the proposed embankment as shown in Figure 2.5-386. The soils encountered in the abutments are discussed in Subsection 2.5.6.3.1. These excavated areas were backfilled with the Type A cohesive fill material obtained from the designated borrow areas.

A slurry trench was constructed between the diversion channel for the North Fork and the areas that were excavated for the baffle dike and the submerged dam to minimize seepage into the excavations. A series of sumps were dug along the outer perimeters of these excavations to collect the minor seepage that occurred. Pumps were used as required to drain the collected water in the sumps.

The groundwater and seepage conditions used in the stability analyses are discussed in Subsection 2.5.5.2.3.3.

2.5.5.2 Design Criteria and Analyses

2.5.5.2.1 Main Dam

The main dam is designed so as to be stable under all conditions of reservoir operation. The stability of the slopes was investigated under the following loading conditions with the factors of safety so stated:

a. end of construction, 1.49

b. steady seepage with normal storage pool (elevation 690.0 feet), 2.06

c. steady seepage with normal storage pool (elevation 690.0 feet) plus 0.1g horizontal earthquake force, 1.44

d. steady seepage with maximum storage pool (elevation 708.0 feet), tailwater at elevation 675.0 feet, 2.02

e. and, sudden drawdown from normal storage pool (elevation 690.0 feet), 1.50.

Figures 2.5-393 through 2.5-397 illustrate the failure surfaces with the minimum factors of safety as determined by the computer program, BISHOP, utilizing the simplified Bishop method. The BISHOP computer program is described in Appendix C.

In the simplified Bishop method, the failure surface is assumed to be an arc of a circle. The factor of safety is defined as the ratio of the moment about the center of the available resisting forces along the failure arc to the moment tending to cause sliding.

A minimum factor of safety of 1.3, 1.5, 1.5, and 1.0, respectively, is required for the loading conditions listed previously in Items a, b, d, and e. Under seismic loading conditions, a minimum factor of safety of 1.0 is required for the loading condition listed previously in Item c.

The analyses indicated that a side slope of 3:1 (horizontal to vertical) is adequate for the upstream and downstream slopes of the main dam to ensure its stability under both static and seismic loading conditions, with conservative factors of safety.

In the downstream slope of the main dam, the piezometric levels in the embankment and foundation materials will increase with time as steady state seepage is established. As this occurs, the pore pressures increase and the effective stresses decrease.

In the analysis, the effective strength parameters for the soils are used; however, the pore pressures are also considered in determining the effective stresses in the dam when computing the soil strength available to resist failure.

The filling of the reservoir causes no change in consolidation stress in the downstream slope of the dam. Therefore, the critical condition will occur when the seepage line is established and the greatest increase in pore pressure and corresponding reduction in effective strength has occurred. The soil properties used in the analysis for the steady state condition were conservatively chosen from the consolidated, undrained triaxial tests.

If the embankment has not consolidated under its own weight such that consolidated, undrained triaxial tests are applicable, the unconsolidated, undrained test results should be used. These properties were used for the end of construction analysis. This condition is the same as considered for the end of construction analysis where the unconsolidated, undrained soil properties were used.

The Salt Creek alluvium in the foundation and the sand drainage blanket in the downstream slope being sand materials should provide drainage in the embankment (Q&R 241.11).

2.5.5.2.2 Submerged Dam and Excavated Slopes of Ultimate Heat Sink

The stability of the slopes of the submerged dam and the excavated slopes of the cut areas of the ultimate heat sink were investigated by applying the Seismic Category I design criteria.

These analyses are based on the static and dynamic properties of the fill material, subsoil, and the bedrock developed through exploratory borings and laboratory tests.

In this analysis, the following loading conditions were considered:

a. submerged dam -

  1. end of construction,

  2. submerged condition under normal cooling lake elevation (690.0 feet),

  3. sudden drawdown condition as a result of breach of the main dam, maximum storage pool elevation (675.0 feet) in the heat sink, and complete and rapid drawdown on the downstream side, and

  4. same loading conditions as in Item 2 plus safe shutdown earthquake force; and

b. excavated slopes -

1. end of construction,

2. submerged condition under normal cooling lake elevation (690.0 feet) in the heat sink plus safe shutdown earthquake force, and

3. sudden drawdown condition from normal cooling lake elevation (690.0 feet) to maximum storage pool elevation (675.0 feet) in the heat sink.

The static stability of the slopes was investigated using the simplified Bishop method, implemented in the SLOPE and BISHOP computer programs, and based on the criteria presented in Subsection 2.5.5.2.3. Dynamic analyses were performed to evaluate the stability of the slopes under seismic loading conditions. These analyses were accomplished with finite-element models which consider nonlinear stress-strain and damping relations and cyclic strength characteristics of the soil. The details of these analyses are given in Subsection 2.5.5.2.4.

The upstream and downstream slopes of the earthfill are protected against erosion by soil cement. The design of the soil cement slope protection is discussed in Subsection 2.5.6.4.1.2.

The natural slopes bordering the ultimate heat sink on the south side generally have a maximum grade of 5:1 (horizontal to vertical), as shown in Figure 2.5-385, Sections E-E, F-F, G-G, and Y-Y. However, in the vicinity of Section G-G, the distance between the bottom of the heat sink and the steep valley wall is approximately 160 feet. At this location, the natural slope from elevation 675 to 690 is approximately 1.5:1 (horizontal to vertical). If a postulated slope failure did occur in this area and the final slope was 30:1 (horizontal to vertical), the material would only fill at the most the inlet as shown in Figure 2.5-384. The sloughed material would extend approximately 130 feet from the toe of the steep slope and would not travel into the heat sink bottom. Therefore, this material would still not reduce the capacity of the heat sink or obstruct the flow path of the circulating water. Also, in the vicinity of Section Y-Y the south side, the slopes are 3:1 (horizontal to vertical) between elevations 690 and 736. This slope is considered in the stability analysis in Subsection 2.5.5.2.3.

The normal pool of the cooling lake will be at elevation 690.0 feet. Under this condition, the slopes of the uplands below elevation 690.0 feet will be completely submerged. The seepage and the zone of saturation will be dependent on the surrounding groundwater conditions.

The stability of the natural slope in the immediate vicinity of the outlet structure was investigated based on the strength characteristics of the soils under saturated conditions.

The static and dynamic stability of the natural slopes bordering the ultimate heat sink were investigated based on the results of the exploratory borings and laboratory tests that were performed. Possible loss of shear strength due to softening of the materials under submerged conditions was also considered in the analysis. The natural steep slopes bordering the ultimate heat sink were graded to ensure slope stability under all loading conditions. This scheme also provides protection against localized sloughing of the slopes. Figure 2.5-385 shows the final graded slopes in the heat sink area.

The intent of Figure 2.5-384 was not to show the limits of the sloughed material but to indicate how the limits would relate to the heat sink configuration.

At section G-G the maximum slope of 15:1 exists; however there is a cove or inlet along the heat sink perimeter at this location, and the toe of this slope is 160 feet from the bottom of the heat sink. If this slope is postulated to fail and slough to a 30:1 slope, the material would move 130 feet and still be 30 feet from the bottom of the heat sink. This would at most only fill the inlet or cove and not encroach on the heat sink perimeter shown in Figure 2.5-384 (Q&R 241.12).

2.5.5.2.3 Static Slope Stability Analyses

Stability analyses were performed using the simplified Bishop method implemented in the SLOPE and BISHOP computer programs. The descriptions of these computer programs are given in Appendix C.

Three critical slopes in the ultimate heat sink were analyzed as follows:

a. The as-built, excavated slope with the excavated toe, Section X-X, at the east side of the ultimate heat sink (UHS) and in the vicinity of the outlet structure.

b. The as-built, excavated slope with the excavated toe, Section Y-Y, at the south side of the ultimate heat sink. Profile used for the analysis was obtained from Borings H-23 and H-40 as shown in Figure 2.5-390.

c. The as-built, excavated slope in the southeast corner, Section H-H, includes fill material placed in an existing valley. The profile used in the analysis was obtained from field data and Borings H-30, P-61, and P-61A.

These three critical sections were selected based on an evaluation of the variables that were relevant to the stability of natural slopes. The variables and loading conditions considered in the analyses are presented in Subsections 2.5.5.2.3.1 through 2.5.5.2.3.8.

2.5.5.2.3.1 Slope Configuration

Several combinations of slope height and slope inclinations are shown in typical cross sections through the natural slopes bordering the ultimate heat sink, Figure 2.5-385. The location of these cross sections are identified in Figure 2.5-384.

Sections J-J, K-K, H-H, and X-X represent the ground profiles on the east side of the ultimate heat sink, in the vicinity of Seismic Category I structures. Sections E-E, F-F, G-G, and Y-Y represent the ground profiles on the south side of the ultimate heat sink.

The variations of slope height above elevation 690.0 feet on the east and south sides, the maximum slope inclination, and the percentage of height where the maximum inclination occurs, are as follows:

SECTION	SLOPE HEIGHT (feet)	MAXIMUM SLOPE	PERCENTAGE OF HEIGHT
East Side			
J-J	45	3.5:1	100
K-K	45	4.0:1	100

X-X	45	3.5:1	100
H-H	45	3.5:1	100
South Side			
E-E	45	5.0:1	89
F-F	32	6.0:1	31
G-G	22	4.0:1	73
Y-Y	36	3.0:1	55

The height of the excavated slopes (7:1, horizontal to vertical) at the north side ranges from 5 to 20 feet.

2.5.5.2.3.2 Subsurface Conditions

An extensive program of soil exploration described in Subsections 2.5.4.3 and 2.5.5.3.2 was undertaken to determine the subsurface conditions in the ultimate heat sink. The soil profiles determined by these investigations are shown in Figures 2.5-387 through 2.5-392. The locations of these profiles are shown in Figure 2.5-384.

Sections P-P and Q-Q show soil profiles through the ultimate heat sink including the baffle dike. Sections X-X, Y-Y, R-R, and S-S show soil profiles along sections through the east, south, and north sides of the ultimate heat sink, respectively.

A representative soil profile for the east and south sides of the ultimate heat sink was developed for use in the stability analysis. The stratigraphic units and elevations shown in the following table were defined based on the soil profiles of Sections P-P, Q-Q, X-X, and Y-Y (Figures 2.5-387 through 2.5-390) and site geologic profiles (Figures 2.5-275, 2.5-279, and 2.5-284), as well as actual field conditions encountered.

	Elevation (feet)	
Stratigraphic Unit	From	To
Richland Loess	740	730
Wisconsinan Glacial Till of the Wedron Formation	730	695
Interglacial Zone of the weathered Glasford Formation and Robein Silt where present	695	680
Sand Layer of the upper Glasford Formation	680	675
Illinoian Glacial Till of the unaltered Glasford Formation	675	570
Lacustrine Deposits of the Banner Formation	570	560
Pre-Illinoian Glacial Till of the Banner Formation	560	480

Bedrock	480

The Richland loess consists of medium to stiff clayey silt or silty clay with trace of fine sand, weathered.

The Wisconsinan glacial till of the Wedron Formation consists of stiff to hard clayey silt and silty clay with sand and gravel-sized particles randomly interspersed throughout the matrix or in well sorted lenses of varying thickness.

The interglacial zone, consisting of the Robein Silt and the weathered Glasford Formation, is composed of stiff to very stiff silt or silty clay, clayey silt, sand, and gravel.

The Illinoian glacial till of the unaltered Glasford Formation is a very hard and dense material and consists of clayey silt with occasional interspersed sand and gravel-sized particles and pockets of fine to coarse sand.

The lacustrine deposits of the Banner Formation contain stiff to hard clayey silt or silt and clay with sand and gravel.

The pre-Illinoian glacial till of the Banner Formation is composed of silty clay and clayey silt with some interspersed sand and gravel.

Bedrock consists of interbedded layers of limestone, shale, and siltstone.

The conservative design effective soil parameters assigned to each stratigraphic unit are described in Subsection 2.5.5.2.3.7.

The stratigraphic units down to and including the Illinoian glacial till of the unaltered Glasford Formation form the natural slope. The remaining stratigraphic units are below the toe elevation of the natural slopes.

The slope configuration above toe elevation (height and inclination) is the governing factor for the stability analysis. Since Section X-X (east side) and Y-Y (south side) had the maximum height and the steepest inclination, those sections were considered the most critical. Section H-H (east side) includes fill material placed above elevation 690 and was considered a separate case to be analyzed.

The soil profiles (Figures 2.5-391 and 2.5-392) at the north side of the UHS consist of alluvial materials of the Henry Formation overlying the Illinoian glacial till. The total alluvial fill, including the topsoil, ranges in thickness from 12 feet to 23 feet. These alluvial deposits include an upper zone, 5 to 10 feet in thickness composed of soft to medium silty clay and/or clayey silt.

2.5.5.2.3.3 <u>Groundwater Table</u>

The groundwater table was established from the elevation shown in geologic Section B-B' (Figure 2.5-275).

2.5.5.2.3.4 Water Elevation in the Ultimate Heat Sink

The elevation of the water surface in the ultimate heat sink and/or cooling lake was established at either elevation 675 or 690, depending on the condition being considered in the stability analyses.

2.5.5.2.3.5 Earthquake Forces

Analyses were performed with and without earthquake forces. When earthquake forces were used for pseudo-static cases, a horizontal ground acceleration of 0.25g was used.

2.5.5.2.3.6 Stability Conditions

Three different conditions were considered in the analyses of the stability of the slopes:

a. end of construction,

b. full cooling lake, and

c. empty cooling lake (rapid drawdown).

End of construction simulates the conditions at the time when the ultimate heat sink (UHS) is completed and the slopes are finished with the excavated toe. There will be no water contained in the UHS.

Full cooling lake simulates the conditions existing during normal operation of the station with water surface established at elevation 690 feet.

Empty cooling lake simulates the conditions existing in the event that the main dam fails and the water contained in the cooling lake is lost, rapid drawdown.

For end of construction and full cooling lake conditions, a steady seepage of groundwater toward the face of the slope was established with a piezometric surface of top flow line. This line defines submergence of the slope where a body of water exists. The groundwater surface is represented by a piezometric surface which calculates the buoyant force on the overburden column of soil. This is referenced to the soil or soils which are affected by this pore pressure.

For the end of construction condition, the piezometric surface was established from the groundwater table shown in geologic Section B-B' (Figure 2.5-275). For full cooling lake condition, the water surface was established at elevation 690 in the ultimate heat sink and the groundwater surface in the slope (for these two conditions, see Figures 2.5-398 and 2.5-399 for Section X-X, Figures 2.5-400 and 2.5-401 for Section H-H, and Figures 2.5-489 and 2.5-490 for Section Y-Y).

The empty cooling lake condition is represented by a drawdown case which is the lowering of the water level against the slope. The initial water level and steady state after drawdown level are defined by two piezometric surfaces, 2 and 1, respectively. The pore pressure calculations are a function of piezometric surfaces 1 and 2 (see Figure 2.5-402 for Section X-X and Figure 2.5-491 for Section Y-Y).

For this case, the initial water level is the lowering of the piezometric surface established for full cooling lake condition against the slope and the drawdown level is 675 feet, the maximum storage pool elevation in the ultimate heat sink.

For Section H-H, the piezometric surface was defined by the anticipated water level immediately following the loss of the cooling lake down to elevation 675 feet as shown in Figure 2.5-403.

Static and pseudo-static analyses were performed for end of construction and full cooling lake cases; only static analyses for the empty cooling lake case. Effective stress parameters were utilized in all cases.

The end of construction condition was not considered for Section H-H because the fill material had been in place for some time prior to the stability analysis of the section and no sloughing of material was noticed.

2.5.5.2.3.7 Selection of Soil Parameters

The effective strength parameters (cohesion and angle of internal friction) for the soil deposits comprising the slopes were obtained from the Mohr's circles plots using the results of consolidated undrained triaxial tests with measurements of pore pressure. These consolidated undrained triaxial test data are shown in Tables 2.5-10, 2.5-11, and 2.5-13, and the Mohr's circles are presented in Figures 2.5-404 through 2.5-408.

The strength parameters for the loess and the sand layer in the upper Glasford Formation were selected from published data showing correlations between number of blow counts and angle of internal friction. The following are the correlations used to obtain the strength parameters.

Soil Type	Average N-Value (blows/foot)	Relative Density	Angle of Friction
Loess	28	NA	20°
Sand	44	75%	38°

The static moduli of elasticity were calculated using the stress-strain curves of the triaxial tests, at the strain corresponding to one-half of the peak deviator stress. The densities shown in Table 2.5-53 are averages of natural densities obtained from numerous samples taken from the borings. The Poisson's ratios are assumed values taken from published data. The soil properties used in the analysis for Section H-H are based on the borings in that area and are provided in Figures 2.5-400, 2.5-401 and 2.5-403.

The design effective strength parameters are shown in Table 2.5-53. Cohesion and friction values correspond approximately to the minimum of the strength parameters of all soils within each stratigraphic unit, based on the data presented in Figures 2.5-404 through 2.5-408.

The blow counts corresponding to the samples used in triaxial compression tests fall within the range of the minimum blow counts for the soil layers within each stratigraphic unit. Consequently, the strength parameters are representative of the weak soils with low blow counts and low density. Blow counts less than those shown in the data of Figures 2.5-404 through 2.5-408 are found in the Wisconsinan till at Boring P-38, (elevation 730), and in the Illinoian till at Boring P-44, (elevation 680 feet). Since the excavation for the construction of the station, screenhouse, and the vicinity of the outlet structure is removed from these soft soils, strength parameters for these soils were not determined.

Based on the conservatism used in the selection of strength parameters, the same cohesion and friction values were assigned to the Wisconsinan till of the Wedron Formation and interglacial cohesive coils of the weathered Glasford Formation. Conservative parameters for the Illinoian till of the unaltered Glasford Formation were chosen for stability analysis (42° angle of internal friction and 1400 lb/ft^2 cohesion). These parameters were selected based on the Mohr's circles plot shown in Figure 2.5-406.

The same properties were assigned to the pre-Illinoian lacustrine or glacial till due to similarity in soil properties.

The soil profile for Section X-X of the Ultimate Heat Sink is shown in Figure 2.5-389. This figure shows the sand layer underlying the Interglacial Zone which is designated as Soil No. 4 in the model used for the stability analysis (Figure 2.5-398). As discussed in Subsection 2.5.5.2.3.9, this sand layer belongs to the upper Glasford Formation and has an average N-value of 44 blows per foot which corresponds to 75% relative density. The strength parameters for this material were selected as ø equal to 38°, c equal to 0 from correlations between number of blow counts and angle of internal friction (Subsection 2.5.5.2.3.7).

The blow counts in the range 2, 4, 5 in borings P-12 and P-8 are for clayey silt (ML) and silty clay (CL) which overlay the sand (SP) soils. The sand materials in these two borings had N-values of 23 and 25 relative densities of 91% and 95% (Table 2.5-54). The clays and silts in borings P-8 and P-12 lie above the elevation 668.5 and were removed during excavation of the heat sink and toe of slope. The material below the excavation is sand with relative densities of 75% or greater for which the ϕ equal to 38°, C equal to 0 strength parameter are applicable.

Published data (H. F. Winterkorn and H. Y. Fang, Foundation Engineering Handbook, Van Nostrand Reinhold Co., 1975) show that for dense sand (N = 30 to 50, relative density = 65% to 85%). The range of ϕ values is 36° to 41° according to Peck and 40° to 45° according to Mayerhoff (Table 2.43 p. 117). A compilation of many relationships between relative density vs. friction angle for cohesionless soils (Figure 7.26, Winterkorn and Fang, p. 263) also shows that for relative densities of 75% and greater, the angle of internal friction is ϕ = 38° or greater (Q&R 241.13).

The slope stability analysis at Section H-H of the Ultimate Heat Sink as shown in Figure 2.5-400 used shear strength parameters derived from samples of the Wisconsinan Till compacted to 90% modified optimum density as given in Table 2.5-13.

During construction of the Main Dam, the embankment was partially completed and left to stand during the winter of 1976-77. At the resumption of work in the spring of 1977, samples were taken from the zone of frost penetration to determine what effects the frost had on the soil properties. These soil properties were used in the Main Dam analysis and are shown in Table 2.5-52. Subsection 2.5.6.9 describes this testing program.

The use of the soil properties for frost affected soil was to show that an adequate factor of safety is available even if the reduced soil parameters were used. The fill in the Ultimate Heat Sink slope was constructed during a single construction season and material interior to the fill was not subject to frost.

The normal pool elevation shown in Figure 2.5-403 is intended only as a reference level from which the drawdown occurred. The water level at elevation 675 feet was used in the analysis (Q&R 241.14a).

2.5.5.2.3.8 Results of Stability Analyses

The results of the stability analyses for as-built excavated slopes are presented in the following Sections (X-X, Y-Y, and H-H) and graphically in Figures 2.5-398, 2.5-399, and 2.5-402 for Section X-X, Figures 2.5-400, 2.5-401, and 2.5-403 for Section H-H, and Figures 2.5-489, 2.5-490, and 2.5-491 for Section Y-Y. The critical circle is drawn on the section the referenced to the center by the radius lines.

FACTORS OF SAFETY

CONDITIONS	SECTION X-X	SECTION Y-Y	SECTION H-H
End of Construction			
Static	2.60	2.42	NOT
Pseudo-Static	1.24	1.21	ANALYZED
Full Cooling Lake			
Static	2.33	2.15	2.32
Pseudo-Static	1.07	1.03	1.02
Empty Cooling Lake			
Static	1.96	2.09	2.16

The stability analysis for Section X-X considered an excavated slope configuration of a 5:1 (horizontal to vertical) slope from the heat sink bottom to elevation 690, and then a 3.5:1 (horizontal to vertical) slope up to natural ground at approximately elevation 736.

The stability analysis for Section Y-Y considered an excavated slope configuration of 5:1 (horizontal to vertical) from the bottom of the heat sink to elevation 690, and then a 3:1 (horizontal to vertical) slope from elevation 690 to elevation 736.

Based on the results of the static and pseudo-static stability analyses, the slopes of the ultimate heat sink are stable with an adequate factor of safety.

At the north side of the ultimate heat sink, the excavated slopes are very flat (7:1, horizontal to vertical). Attachment A2.5 discusses this slope in greater detail with regard to slope stability.

The submerged dam at the west side of the ultimate heat sink, with a maximum height of 21 feet and side slopes of 5:1 (horizontal to vertical), was constructed using Wisconsinan glacial till of the Wedron Formation (Type A material). Static analyses performed for the main dam having a maximum height of 57 feet and side slopes of 3:1 (horizontal to vertical) constructed with Wisconsinan till of the Wedron Formation (Type A material), show adequate factors of safety. Based on these analyses, it is concluded that the slopes of the submerged dike are also stable under an adequate factor of safety. Finite-element analyses are presented in Subsection 2.5.5.2.4.

2.5.5.2.3.9 Liquefaction Potential

From the point of view of liquefaction, the alluvial soils of the Henry Formation on the floodplain and the sand layer in the upper Glasford Formation on the valley walls are of interest. The presence of these soils was determined by the borings located in Figure 2.5-16.

The simplified procedure for evaluating soil liquefaction described by Seed and Idriss (Reference 85) has been applied to the alluvial materials which have a relatively level surface. The finite-element dynamic analysis described in Subsection 2.5.5.2.4.4 incorporates the sand layer behind the natural slopes.

The alluvial soils at the east side of the ultimate heat sink were evaluated from Borings P-5, H-28, H-32 (vicinity of the screen house) and Borings P-8, P-12, and H-33 (vicinity of the outlet structure). From the results shown in Table 2.5-54, it is seen that the soils having relative densities greater than 90% are safe against liquefaction. Borings H-28, H-32 and H-33 indicated the presence of some loose alluvial material above elevation 667 feet. The excavation for the screen house and outlet structure removed this material (Figures 2.5-379 and 2.5-381). The excavated areas around the screen house and outlet structures were backfilled with either compacted Wisconsinan till of the Wedron Formation (Types A and C material) or fly ash backfill. The extent of the backfill materials is discussed in Subsection 2.5.4.5.2 for the screen house, and Subsection 2.5.4.5.3 for the outlet structure. The alluvial material in other borings in this area have relative densities in excess of 90% with factors of safety against liquefaction ranging from 1.01 to 1.19

The sand layer in the upper Glasford Formation revealed by Borings P-9, P-13, P-37, P-48, H-30, and H-31 (east side) and Borings H-23 and H-26 (south side) has an average N-value of 44 blows per foot, which corresponds to 75% relative density. This deposit consists of clean, silty sand and gravel of varying gradation. The dynamic analysis performed for Section X-X (the most critical subsoil profile discussed in Subsection 2.5.5.2.3.2) shows that the elements of the sand layer are safe against liquefaction. This is described in Subsection 2.5.5.2.4.

It is concluded, therefore, that the excavated slopes, the backfill, and the alluvial materials in the vicinity of the screen house, outlet structure, and the east side of the heat sink will maintain their integrity and not liquefy during the safe shutdown earthquake.

Along the north and south sides of the periphery of the ultimate heat sink, a flow-slide method of analysis was used to determine the final slope conditions after the safe shutdown earthquake is postulated to have caused flow-type slides of the side slopes.

Attachment A2.5 discusses four postulated failure modes that could possibly reduce the capability of the UHS to function normally.

To verify that adequate capacity remains in the heat sink following any local seismic activity with acceleration greater than the OBE, the bottom of the heat sink will be monitored as described in Subsection 2.4.11.6.

2.5.5.2.4 Dynamic Slope Stability Analysis

2.5.5.2.4.1 Method of Analysis

The stability of the submerged dam and the natural slopes under seismic loading conditions was investigated using the dynamic finite element analyses which consider the strain-dependent material properties. For the dynamic analyses, the postulated safe shutdown earthquake presented in Subsection 2.5.2.6 was used. The loading conditions for which the dike and natural slope were analyzed are discussed in Subsection 2.5.5.2.2. The static and dynamic properties of the fill material and foundation soils were based on a comprehensive soil exploration and laboratory testing program as discussed in Subsection 2.5.4.2.

The procedure used in evaluating the seismic stability of the slopes consists of the following steps:

a. determination of the response of the dam foundation system to the compatible rock accelerations, including the evaluation of the induced shear stresses at various locations throughout the dam and the foundation material;

b. representation of the irregular cycles of shear stresses induced in the dam foundation system by an equivalent number of cycles of equivalent uniform shear stresses;

c. determination of the static stresses existing in the dam foundation system (prior to the rock acceleration);

d. determination of the cyclic shear stresses required to cause single amplitude shear strains greater than 5×10^{-2} in the material for conditions representative of those existing in the dam foundation system by means of appropriate cyclic load tests on representative specimens of the materials; and

e. evaluation of the seismic stability of the dams by comparing the shear stresses required to cause single amplitude shear strain greater than 5×10^{-2} with the equivalent shear stresses induced by rock accelerations.

2.5.5.2.4.2 Step-By-Step Procedure Used in Seismic Stability Evaluation

The following steps were used in evaluating the seismic stability of the dike and the natural slope:

1. Generation of a Compatible Time-History, G(t)

Synthetic accelerograms were generated for horizontal and vertical ground motions such that the response spectra of these accelerograms envelope the design response spectra. For this purpose, the N-S component of the 1940 El Centro earthquake record was suitably modified using the RSG program described in Appendix C. The normalized accelerograms are shown in Figures 2.5-409 and 2.5-410 for the horizontal and vertical ground accelerations, respectively. The close matching of the response spectra obtained for the synthetic accelerogram with the design response spectra is demonstrated in Figures 3.7-1 through 3.7-10.

2. Generation of a Compatible Horizontal Rock Motion, I_H (t)

The soil profile below the toe of the slopes was modeled as a continuous shear layer system. The compatible time-history for the horizontal ground motion as obtained in Step 1 was input at the top layer of this model. A maximum acceleration of 0.25g was used. The soil properties were defined in terms of the curves for strain dependent soil moduli and damping ratios. By inputting the compatible time-history for horizontal ground motion and iterating on the strain dependent soil properties, a compatible horizontal rock motion was obtained using the computer program SHAKE, described in Appendix C.

3. Generation of a Compatible Vertical Rock Motion, I_V (t)

The vertical rock motion was obtained by exciting the SHAKE model used in Step 2 with the compatible time-history for the vertical ground motion. A maximum acceleration of 0.25g was used. The soil properties used for this step were corresponding to the properties obtained in Step 2. The vertical rock motion I_V (t) thus obtained was used as input at the rock level in the two dimensional model used in Step 4.

4. Dynamic Response Analysis

The horizontal rock motion I_H (t) and the vertical rock motion I_V (t) were used simultaneously, for the dynamic response analysis of the dike and the slope to evaluate the shear stress time-histories at various locations in the dike and slope. The response computation was performed using the finite element method of analysis. The computer program QUAD-4, described in Appendix C, was used to compute the response in the embankment foundation system. This program incorporates the use of strain-dependent modulus and damping ratio for each element of the model. The width of the model was kept large enough to ensure free field conditions at the remote boundary. Several iterations were made on the soil properties and finally, the shear stress time-history generated in each element as a result of the simultaneous horizontal and vertical rock motion was obtained.

5. Representation of Irregular Shear Stress Time-History by Equivalent Uniform Shear Stress

The procedure used to represent the irregular shear stress time-history at any element by equivalent uniform shear stress corresponding to any N_c number of cycles, involves the following operations:

a. sorting the peak stresses, saving only the largest stress value between each zero crossing (both positive and negative peak values are saved);

b. rearranging the peak shear stresses in descending order so that the largest peak is first and the smallest is last;

c. averaging of the absolute values of the positive and negative stress peaks for each cycle; and

d. the cumulative average shear stress is computed for increasing number of cycles. The cumulative average value corresponds to the equivalent shear stress for the N_c number of cycles for which the average has been obtained.

6. Static Stress Analysis

A knowledge of the initial static effective stress conditions is required for the evaluation of the cyclic strength of materials in the dike and the slope. For the dike, an incremental finite-element approach was used which simulates the construction of an embankment in a series of layers. The dike was divided into several horizontal layers each represented by quadrilateral elements. During any increment of the layer, appropriate values of modulus E and Poisson's ratio ν were assigned to each element. After determining the stresses, E and ν were reevaluated for the average stress conditions during the new increment and compared with the assigned values. If a significant difference was obtained, the E and ν values were adjusted until a reasonable correspondence was established between the input and the computed values. This process was continued until the last layer was added. The effect of buoyancy on stresses was evaluated by using submerged unit weights for all materials in the dike and its foundation. The analysis was conducted using the computer program EMBANK, described in Appendix C. For determining stresses in the foundation of the dike and the entire natural slope, the weight of all the materials of the slope was turned on simultaneously, and the stresses were determined assuming the at-rest condition.

7. Dynamic Material Properties

In order to conduct the analysis, the cyclic shear stresses required to cause single amplitude shear strains greater than 5% in all the materials for conditions representative of those existing in the slope, dike, and the foundations must be determined. The data presented in Figures 2.5-411 and 2.5-412 have been derived from dynamic triaxial compression test results on recompacted Wisconsinan till of the Wedron Formation, Type A material, conducted in accordance with the procedures described by Seed and Peacock (Reference 94). It has been assumed that the data are applicable to interglacial zone, Wisconsinan till of the Wedron Formation, compacted fill of Wisconsinan till of the Wedron Formation (Type A material) and Illinoian till of the unaltered Glasford Formation. Based on an evaluation of strength parameters given in Tables 2.5-55 and 2.5-53, respectively, it is anticipated that compacted Wisconsinan till of the Wedron Formation, Type A material, is likely to exhibit minimum dynamic strength as compared with the strength values for other materials involved herein. Therefore, it appears to be conservative to use the dynamic properties of compacted Wisconsinan till of the Wedron Formation, Type A material, for representing the material properties of the four materials.

The dynamic material properties given in Figure 2.5-413 refer to the in situ interglacial sand deposits overlying the Illinoian till, of the unaltered Glasford Formation, and were obtained from appropriate dynamic triaxial compression tests.

8. Evaluation of Seismic Stability

The minumum factor of safety for various elements against local failure due to seismic loading was determined by comparing the shear stresses required to cause single amplitude shear strains greater than 5% with the equivalent shear stresses induced by the simultaneous action of horizontal and vertical rock accelerations. The induced equivalent uniform shear stresses were determined using the procedure described in Step 5. A minimum value of this stress ratio greater than approximately 1.1 is generally considered to provide an ample margin of safety.

2.5.5.2.4.3 Description of the Submerged Dike and Natural Slopes

The geometry of the cross sections of the submerged dike and that of the natural slope analyzed for the seismic stability are shown in Figures 2.5-414 and 2.5-415, respectively. Based on the height, the steepness of the slopes, the depth to bedrock and the soil data for the site, these two cross sections were considered to be the most critical sections for the dike and the east side slope of the ultimate heat sink (UHS) in the vicinity of Seismic Category I structures. These are referred to as maximum cross sections. Due to construction features, the excavated slopes in the vicinity of the screen house and outlet structure are now 5:1 (horizontal to vertical) for the slope below elevation 690 feet. Thus these slopes are flatter than the natural slope cross section used in the analysis. This flattening of the slopes will add safety to the slopes. The submerged dike has a maximum height of approximately 17 feet with a side slope of 5:1 (horizontal to vertical) for both the upstream and downstream slopes. It has been constructed using the Wisconsinan glacial till of the Wedron Formation, Type A material, excavated from borrow areas. The maximum cross section of the submerged dike and the soil layering under the dike down to the bedrock shown in Figure 2.5-414 is based on Boring H-6.

For the soils forming the natural slope and the foundation of the submerged dike, the material properties are based on laboratory test results, field measurements, and published data.

2.5.5.2.4.4 Seismic Stability Evaluation of Submerged Dike

The maximum cross section of the submerged dike is shown in Figure 2.5-414. The soil profile of the material under the dike is modeled as a continuous shear layer system as shown in Figure 2.5-416. The strain dependent shear moduli and damping ratio values for all the materials of the dike and foundation are given in Table 2.5-48.

To conduct the dynamic response analysis, a finite element model of the dike, shown in Figure 2.5-417, was prepared. This model was excited at its base with the horizontal and vertical rock motions, I_H (t) and I_V (t). A typical stress time-history induced in an element is presented in Figure 2.5-418. The procedure used for representing the irregular shear stress time-history by equivalent uniform shear stress has been illustrated in Table 2.5-64.

The soil properties used in the static stress analysis of the dike are given in Table 2.5-55. The finite element model used for this analysis is similar to that being used for the dynamic analysis; however, it was necessary to use only half the model for static analysis because of the symmetry. The boundary conditions at the line of symmetry were such that only vertical displacements were permitted along this boundary of the model.

To evaluate the factors of safety for various elements in the dike, the shear stresses required to cause 5% single amplitude shear strain in the elements were compared with the shear stresses induced in the elements due to earthquake (Step 8 in Subsection 2.5.5.2.4.2). Such a comparison is presented in Table 2.5-56. It will be seen from this table that the minimum factor of safety against development of 5% single amplitude shear strain exceeds the minimum required factor of safety of 1.1 for all the elements analyzed.

2.5.5.2.4.5 Seismic Stability Evaluation of Natural Slope

The maximum cross section for the natural slope, as used in the analysis, is shown in Figure 2.5-415. In order to obtain the horizontal and vertical rock motion, Steps 2 and 3 in Subsection 2.5.5.2.4.2 were used. The continuous shear layer model is shown in Figure 2.5-419. The variation of shear moduli and damping ratio values with strain for the materials in the slope and its foundation is given in Table 2.5-48. The dynamic response analysis was conducted using the finite element model shown in Figure 2.5-420. The irregular shear stress time-histories obtained for various elements of the model were represented by equivalent and uniform shear stresses of equivalent corresponding number of cycles using the procedures described in Step 5 in Subsection 2.5.5.2.4.2. The static stresses in various elements were obtained using the procedure described in Step 6 of Subsection 2.5.5.2.4.2. The soil properties used in the analysis are given in Table 2.5-53. The evaluation of seismic stability was done by determining the local factor of safety for various elements. The results are summarized in Table 2.5-57. It can be determined from this table that the minimum factor of safety against developement of 5% single amplitude shear strain in the natural slope exceeds the minimum required value of 1.1 for all elements analyzed.

2.5.5.2.4.6 Seismic Stability Evaluation of Submerged Dike and Natural Slope for a New Madrid Type Event

In addition to evaluating the effect of safe shutdown earthquake, the stability of the submerged dam and the natural slopes under seismic loading conditions was also investigated for a New Madrid type event, as specified in Reference 116. A 90 seconds-long time history was developed by modifying the April 1949 Western-Washington earthquake recorded at Olympia (Washington highway test lab, N86E component) to envelope the four-response spectra due to New Madrid type event shown in Figures 35 through 38 of Reference 116. This time history was then used for the stability analysis.

The procedure used for stability analyses of earth-structures is essentially similar to that described in Subsection 2.5.5.2.4.1. However, only the horizontal component of the ground motion has been considered and the motion is applied at the ground surface. The results are summarized in Tables 2.5-58 and 2.5-59. It can be seen from these tables that the minimum factors of safety for all the analyzed elements of the dike as well as of the natural slope exceed the minimum required safety factor of 1.1.

2.5.5.2.4.7 Conclusion

Based on the finite elements method of analysis and using the procedure described, it can be concluded that the minimum local factor of safety in the cross sections of the submerged dike as well as of the natural slope exceed the allowable minimum factor of safety for the embankment and the facilities surrounding the ultimate heat sink will maintain their integrity under the safe shutdown earthquake and a New Madrid type event.

2.5.5.3 Logs of Borings

2.5.5.3.1 Main Dam

Borings in the borrow area for the main dam are located in Figure 2.5-272. The logs of the borings are presented in Figures 2.5-129 through 2.5-144.

Laboratory testing was performed on representative bulk samples of the borrow material. Results of the tests are presented in the following tables and figures:

a. optimum moisture contents: Table 2.5-52;

b. particle size analyses of placed material: Figure 2.5-421;

c. field compaction tests: Figure 2.5-422;

d. consolidation tests: Figures 2.5-332, 2.5-336, and 2.5-337;

e. triaxial compression tests: Tables 2.5-12 and 2.5-13;

f. unconfined compression tests: Tables 2.5-8 and 2.5-9;

g. Atterberg limit of placed material: Table 2.5-52; and

h. permeability tests: Table 2.5-33.

A summary of the properties of the materials used to construct the main dam is presented in Table 2.5-52.

2.5.5.3.2 Ultimate Heat Sink

Borings in the borrow areas and the vicinity of the submerged dam and baffle dike are located in Figure 2.5-16. The logs of the borings are presented in Figures 2.5-162 through 2.5-221.

Laboratory testing was performed on representative bulk samples of the borrow materials and in situ material. Results of the tests are presented in the following tables and figures:

a. optimum moisture contents: Table 2.5-36;

b. particle size analyses of placed material: Figure 2.5-424;

c. field compaction tests: Figure 2.5-423.

d. triaxial compression tests: Tables 2.5-11 and 2.5-17; Figures 2.5-404 through 2.5-406 and 2.5-408;

e. unconfined compression tests: Figures 2.5-204, 2.5-205, 2.5-207, 2.5-210 through 2.5-214, and 2.5-216 through 2.5-221;

f. dynamic triaxial compression tests: Table 2.5-22; Figures 2.5-411 and 2.5-412;

g. resonant column tests: Table 2.5-27;

h. Atterberg limit of placed material: Tables 2.5-60 and 2.5-36; and

i. permeability tests: Tables 2.5-31 and 2.5-33.

2.5.5.3.3 Section E-E' along the North Fork of Salt Creek

Borings in the vicinity of Section E-E' along the North Fork of Salt Creek are located in Figure 2.5-278. The logs of the borings are presented in Figures 2.5-94 to 2.5-97. Results of laboratory testing of samples are presented in Tables 2.5-6, 2.5-23, and 2.5-43.

2.5.5.4 Compacted Fill

2.5.5.4.1 Main Dam

The specifications for the placement of compacted fill for the main dam are presented in Subsection 2.5.6.4.2.1.

2.5.5.4.2 Ultimate Heat Sink

The slopes on the north and south sides of the ultimate heat sink were excavated with no compacted fill material being placed in these areas. The west end of the heat sink is defined by the submerged dam. The compacted fill specifications and construction procedures for the embankment are discussed in Subsection 2.5.6.4.1.1. Compacted fill was also placed at the abutments for the submerged dam and is discussed in Subsection 2.5.6.3.1. A 2-foot thick, 8-inch layered system of soil cement slope protection was provided for the submerged dam, and at the abutments of the submerged dam up to elevation 700 feet. A 3-foot thick layered system of soil element was placed on top of the baffle dike. A typical cross section of the embankment and slope protection is shown in Figure 2.4-24.

The east side of the ultimate heat sink is comprised of the screen house and backfill, the baffle dike abutment, the SSWS outlet structure, and the natural excavated slopes. Compacted fill consisting of the Wisconsinan glacial till of the Wedron Formation was used around the screen house and outlet structure as backfill material in the overexcavated areas for these structures. This operation is discussed in Subsections 2.5.4.5.2.5 and 2.5.4.5.3.5. Compacted fill used for the baffle dike abutment consisted of the same material used to construct the baffle dike as discussed in Subsection 2.5.6.4.1. The natural slopes comprising the remaining portions of the east side were excavated where necessary to provide final slopes of 5:1 (horizontal to vertical) up to elevation 690 and then slopes of 3.5:1 (horizontal to vertical) to natural grade.

2.5.6 Embankments and Dams

2.5.6.1 General

2.5.6.1.1 Ultimate Heat Sink

The emergency core cooling system ultimate heat sink was formed by constructing a submerged pond in the valley of the North Fork of Salt Creek. The submerged pond is located immediately west of the screen house and main plant as shown in Figure 2.5-386. The pond's required capacity was developed by constructing an earth dam across the valley bottom and excavating a portion of the valley bottom down to a bed elevation of 668.5 feet. An earthfilled dike was constructed down the center of the pond to lengthen the flow path of the emergency cooling water.

The top elevation of the earth dam is 675 feet, and 675 feet for the baffle dike. With the normal operating water level in the lake at elevation 690 feet, the earthfill will be submerged and will not be in use. However, when the water level in the lake drops below elevation 675 feet, the submerged dam will act as an overflow weir and maintain a water level of 675 feet at all times at the intake screen house. The submerged dam was designed to impound the quantity of water necessary for a 30-day supply in order to safely shut down the reactor in the event that the lake level drops below elevation 675 feet.

2.5.6.1.2 Main Dam

The main dam is an earthfill structure constructed to impound the water for the cooling lake. It is located approximately 1200 feet downstream of the confluence of the North Fork of Salt Creek and Salt Creek. The ground surface elevation of the valley bottom at the dam site is approximately 655 feet. The crest of the dam is at 711.8 feet. The normal operating water level in the cooling lake is elevation 690 feet with a probable maximum flood (PMF) level at elevation 708.8 feet.

An 80-foot wide concrete service spillway is located on the west side of the dam near the west abutment. The ogee of the service spillway has a crest elevation of 690 feet. In addition to the spillway, there is a low-water intake structure designed to maintain a constant flow downstream of the dam if the lake level falls below elevation 690 and to lower the water level in the lake down to elevation 668 if necessary.

On the east side of the main dam, a 1200-foot wide auxiliary spillway was excavated. It was from this area that the material used to construct the dam was obtained. The auxiliary spillway was designed to be operational when the lake level exceeds 700 feet. Figures 2.4-1 and 2.4-13 give a plan view of the dam site illustrating the location of the dam and its appurtenant structures.

2.5.6.2 Exploration

2.5.6.2.1 Ultimate Heat Sink

A total of 60 test borings were drilled in the heat sink area as shown in Figure 2.5-16, and described in Subsections 2.5.4.3 and 2.5.5.1.2. The logs of the exploratory borings are shown in Figures 2.5-162 through 2.5-221.

The exploration program in the ultimate heat sink showed that the North Fork of Salt Creek has eroded into the Illinoian till of the unaltered Glasford Formation. During glacial and postglacial times, the original stream valley has been filled with alluvial silt and silty clay and outwash sand and gravel. The existing stream flowed on this alluvial and outwash valley fill material. The exploration program indicated that these glaciofluvial deposits ranged in thickness from 12 to 23 feet. The upper 5 to 10 feet consisted of silty clay and clayey silt. The lower part consisted predominantly of medium to coarse sand.

Illinoian till underlies the valley fill deposits. The Illinoian till consists of hard, dense, sandy silt and clayey silt and occasional seams and pockets of sand and gravel. The Illinoian till is underlain by pre-Illinoian glacial deposits which are comprised principally of clayey silt and sandy silt. Under the southwest corner of the ultimate heat sink, the pre-Illinoian till is underlain by Mahomet Bedrock Valley deposit. The pre-Illinoian till and the glacial outwash in the southwest part of the ultimate heat sink are underlain by Pennsylvanian bedrock. Pennsylvanian bedrock at the ultimate heat sink consists primarily of shales and siltstones. Geologic sections are shown in Figure 2.5-284.

2.5.6.2.2 Main Dam

A total of 60 borings along the dam alignment and 17 borings in the borrow area were drilled to investigate the subsurface conditions at the dam site. The boring locations are shown in Figure

2.5-272, and a discussion of the drilling operation is presented in Subsection 2.5.4.3. The logs of the exploratory borings are shown in Figures 2.5-74 through 2.5-144.

The location of the dam is situated on the floodplain of Salt Creek where glacio-fluvial deposits have filled the original channel cut by the river in the Illinoian till. Wisconsinan through Illinoian age glacial materials form both of the dam abutments.

The channel fill was from 16 to 32 feet thick (elevation 627 to 660) where penetrated by borings. Topsoil and alluvial silt and clay ranging from 7 to 18 feet in thickness comprised the upper portion of this material. The lower portion of the alluvial material contained principally outwash sand and gravel interbedded with subordinate zones of silt or clayey silt.

The Illinoian till beneath the floodplain deposits, approximately 140 feet in thickness, was composed principally of hard, dense, sandy silt and clayey silt, and occasional intervals of thin layers of sand and gravel. A thin sandy zone in the till was penetrated in Boring D-8 at a depth of 51 feet (elevation 605) or 22 feet below the bottom of the alluvial channel. As presented in Boring D-11, pre-Illinoian glacial deposits, approximately 20 feet in thickness, underlay the Illinoian till. Underlying the pre-Illinoian is a glacio-fluvial sand also about 140 feet thick which comprises the outwash fill of the Mahomet Bedrock Valley. Boring D-11, located 1000 feet downstream from the axis of the main dam, penetrated 50 feet of bedrock consisting of Pennsylvanian limestone, shale, and micaceous siltstone below the valley fill.

2.5.6.2.3 Cooling Lake Reservoir

In general, the cooling lake reservoir basin is enclosed by relatively impermeable glacial till. Above elevations ranging from 640 to 700 feet, till of Wisconsinan age and the interglacial zone form the reservoir sidewalls. Below these elevations, Illinoian till prevails and is composed principally of hard, dense, sandy silt and clayey silt, and occasional intervals of permeable sand and gravel a few feet thick. These permeable intervals are not laterally extensive and, therefore, will not provide significant paths for reservoir seepage. The Illinoian till is generally on the order of 150 feet thick. Except for occasional thin and discontinuous lenses of sand, it is low in permeability. The channels of Salt Creek and the North Fork of Salt Creek, where drilled, have incised the Illinoian till to a maximum depth of about 35 feet (elevation 625). Hence, even beneath the stream channels, at least 115 feet of relatively impermeable Illinoian till will separate alluvial channel deposits beneath the reservoir from the deeper Mahomet Bedrock Valley fill of glacial outwash. The highest elevation of the outwash observed in borings is 500 feet. The outwash is underlain by Pennsylvanian bedrock consisting primarily of shales and siltstone from elevation 360 to 400 feet.

2.5.6.3 Foundation and Abutment Treatment

2.5.6.3.1 Ultimate Heat Sink

Both the baffle dike and the submerged dam in the ultimate heat sink were founded on sound Illinoian till. The bases of the baffle dike and the submerged dam were founded on the till at approximately elevations 660 feet and 658 feet, respectively. All sand pockets exposed at the surface of the submerged dam excavation were removed. Sand pockets exposed at the surface of the baffle dike excavation were tested and confirmed to have 85% relative density or removed. A comprehensive subgrade testing program under the supervision of the Quality Control personnel at the site was instituted to verify that adequate subgrade was obtained.

At the location of the north abutment for the submerged dam, the in situ material was excavated to a final slope of 1.5:1 (horizontal to vertical) from the base of the excavation at approximately elevation 660 feet up to elevation 675 feet. Illinoian till was exposed on this excavated face. Above elevation 675, the in situ material was excavated on a slope of 2:1 (horizontal to vertical) up to existing grade. Following the completion of the excavation, Type A cohesive fill material was placed and keyed into the exposed abutment in near horizontal lifts as described in Subsection 2.5.6.4.1.1.

Likewise, the south abutment for the submerged dam was excavated into in situ material on a slope of 1:2 (horizontal to vertical) to expose Illinoian till between the base of the excavation at approximately 658 feet up to elevation 675 feet. Above this elevation, the in situ material was excavated to a slope of 3:1 (horizontal to vertical) up to existing grade. Type A cohesive fill material was then placed and keyed into the exposed surface in near horizontal lifts.

The east abutment for the baffle dike was excavated from the base of the excavation at approximately 660 feet on a slope of approximately 1.5:1 (horizontal to vertical) up to existing grade. A sand lens in the Illinoian till, averaging 6 feet in thickness and ranging up to 14 feet in thickness, was exposed in the abutment. Type A cohesive material was placed and compacted against the sand in near horizontal lifts. Areas adjacent to the abutment where this sand lens was exposed received a blanket of Type A cohesive material to stabilize the slope and control seepage.

No additional foundation or abutment treatment was performed other than that which was described above. The Illinoian till provided a firm, impermeable base for the foundations of both the baffle dike and submerged dam as well as the two abutments for the submerged dam. This, coupled with the relatively impermeable Type A cohesive material used in the embankments, should prevent any appreciable amount of seepage either beneath the baffle dike or under and around the submerged dam.

2.5.6.3.2 <u>Main Dam</u>

The foundation for the main dam consisted of both the Illinoian till and the lower alluvial materials consisting primarily of medium dense to very dense sand with some gravel. Along the centerline of the dam and extending a minimum of 20 feet each side of the centerline, the Illinoian till was exposed and established as the foundation for the key trench. This key trench was excavated to a minimum depth of 2 feet into the Illinoian till to provide a seepage cutoff for the dam. For the wing areas of the dam, the dense sand was used for the foundation. The sand was considered as acceptable subgrade providing it had a minimum relative density of 70%. However, in some areas, the alluvial sands were not suitable as subgrade material and were excavated to expose the sound Illinoian till. A comprehensive subgrade testing program verified that acceptable subgrade was obtained, that Illinoian till was exposed in the keyway, and that the sand had a minimum relative density of 70% as determined by ASTM D-2049. Figure 2.4-14 shows a typical section of the main dam.

The west abutment was comprised of loess, Wisconsinan till, Robein Silt, and weathered and unweathered (unaltered) Illinoian till. These materials have a stiff to very stiff consistency and possess high shear strength. The abutment was excavated on a slope of approximately 1:1 (horizontal to vertical) through these materials down to the Illinoian till of the unaltered Glasford Formation. The service spillway is located adjacent to the west abutment of the dam and was founded on unweathered Illinoian till. A comprehensive program of subgrade testing was performed as part of the Quality Control program to verify that acceptable subgrade was obtained.

Type A cohesive material was placed and compacted to a minimum dry density of 102% of Standard Proctor on the subgrade beneath the service spillway to bring the foundation up to the design grade. This material was also placed and keyed into the abutment to provide a seepage cutoff.

The east abutment was also comprised of loess, Wisconsinan till, Robein silt, and weathered and unweathered (unaltered) Illinoian till. These materials have a stiff to very stiff consistency and high shear strength. The key trench, 40 feet in width, was excavated through the subgrade of alluvial sands exposed in the wing areas of the dam and into the Illinoian till and extended to the abutment. Illinoian till was exposed in the bottom of the key trench. The interglacial materials and the Wisconsinan till were exposed on the face of the abutment slope which was approximately 3:1 (horizontal to vertical). These in situ materials were considered to be suitable subgrade and Type A cohesive material was placed and keyed into them.

2.5.6.4 Embankment

2.5.6.4.1 Ultimate Heat Sink

The submerged dam is a homogeneous earth embankment constructed using Type A cohesive fill material. The submerged dam has an average height of 17 feet from the subgrade to the top of the dam at elevation 675 feet. The slopes of the dam are 5:1 (horizontal to vertical). A soil-cement mixture was placed in a layered system over the sides and top of the embankment to a thickness of 2 feet to provide slope protection. A typical section of the submerged dam is illustrated in Figure 2.4-24.

The baffle dike is also a homogeneous earth embankment constructed using Type A cohesive material. The baffle dike has an average height of 16 feet from the subgrade to the top of the dike at elevation 676 feet. The side slopes of the dike are 5:1 (horizontal to vertical). The upper 3 feet of the dike embankment is a layered system of soil cement for protection against wave action. A typical section of the baffle dike is illustrated in Figure 2.4-24.

2.5.6.4.1.1 Subgrade and Embankment Materials

The foundation material for both the submerged dam and the baffle dike was the Illinoian till of the unaltered Glasford Formation. A comprehensive program for subgrade testing was initiated to verify that the in-place density was greater than 120 pcf. One test was performed for every 10,000 ft^2 of exposed subgrade for the submerged dam and 20,000 ft^2 for the baffle dike. In addition to verifying in-place density, a laboratory sample for classification characteristics was taken. Laboratory testing on subgrade samples taken at each location included a grain-size analysis (ASTM D-422) and Atterberg limit (ASTM D-423 and ASTM D-424).

Fill placement for the embankments commenced immediately following subgrade approval. The fill material was classified as Type A cohesive fill material. This material was excavated from the borrow areas outlined in Figure 2.5-384. The material was placed in near horizontal lifts not exceeding 8 inches in loose thickness and compacted by a Hyster C-450 segmented pad sheepsfoot roller. After the completion of each day's work, the top of the embankments, borrow areas, and stockpiles were bladed or rolled smooth and crowned slightly to allow rain to freely run off the surface and prevent ponding.

A comprehensive laboratory and field testing program was established as an integral part of the Quality Control program at the site. Representative bag samples of the borrow material were taken for every 6000 yd^3 of material removed to determine the physical characteristics of the material in the laboratory. The following is a list of the tests performed and figures and tables illustrating the average properties of the material used in the embankments:

a. grain-size analysis (ASTM D-422) - Figure 2.5-424;

b. Atterberg limits (ASTM D-423 and D-424) - Table 2.5-36;

c. Modified Proctor density tests (ASTM D-1557) - Figure 2.5-423; and

d. soil classification (ASTM D-2487 and D-2488).

The appropriate specifications for each property are provided on the figures and tables listed above. The Modified Proctor density test was performed once per every 6000 yd^3 of material used.

The field testing program consisted of in-place density and moisture testing using either the Washington densometer method (ASTM D-2167) or the nuclear density method (ASTM D-2922). The in-place density and moisture requirements were a minimum of 90% of the Modified Proctor maximum dry density and a placed moisture content within 4.5% above the optimum moisture content and 2.0% below the optimum moisture content. The frequency of testing was one test per 10,000 ft^2 per lift.

Statistical analyses of the in-place density tests for the Category I cohesive fill used to construct the UHS Dam and Baffle Dike were performed to verify the compaction requirements. The average dry density, average moisture content, and average percent compaction of the fill were determined using the results of the nuclear and sand cone tests performed on the cohesive fill. These averages were computed for each one-foot interval and for the total embankment for both the UHS Dam and the Baffle Dike.

A total of 1,526 in-place density tests were analyzed for the UHS Dam. The average dry density of entire UHS Dam embankment was 125.7 PCF placed at an average moisture content of 10.6%. This dry density corresponds to an average of 93.4% compaction as determined by ASTM D1557.

Figure 2.5-451 shows the distribution for the dry density test results. The lowest in-place dry density recorded was 119.4 PCF while the highest was 139.4 PCF. Figure 2.5-452 shows the distribution for the in-place moisture content of the tests. The lowest recorded value was 5.7% and the highest was 14.1%. Figure 2.5-453 shows the distribution of the percent compaction for the in-place tests. The lowest value recorded was 89.1% and the highest value was 103.4%.

There were a total of 9 in-place tests that did not meet the 90% degree of compaction requirement. This represents approximately 0.6% of all of the in-place tests taken for the UHS Dam embankment. Their values ranged from 89.1% to 89.9%. Two of the nine tests had both a nuclear test (ASTM D2922) and a balloon test (ASTM D2167) performed at the same location. In both cases, one of the two tests met the 90% compaction requirement. Two other tests had values of 89.9% compaction. These 9 tests represent isolated areas within the embankment and have in-place compaction densities only slightly less than the minimum requirement. Therefore, they will not be detrimental to the integrity of the UHS Dam embankment.

There were a total of 5 in-place tests for the submerged dam for which the placement moisture criteria was not met. This represents approximately 0.3% of all the tests. Of these tests, three had moisture contents 0.1% above the specified limit and the other two were 0.2% and 0.6% above the specified limit. These tests represent only isolated areas and are only slightly outside of the specified limit. Therefore, they were accepted and considered not to be detrimental to the integrity of the dam.

A total of 2034 in-place density tests were analyzed for the Baffle Dike. The average dry density of the entire Baffle Dike embankment was 125.6 PCF placed at an average moisture content of 10.8%. This dry density corresponds to 93.6% compaction as determined by ASTM D1557.

Figure 2.5-454 shows the distribution for the dry density test results for the Baffle Dike. The lowest in-place dry density recorded was 116.6 PCF, while the highest was 143.6 PCF. Figure 2.5-455 shows the distribution for the in-place moisture content of the tests. The lowest recorded value was 6.1% and the highest was 14.8%. Figure 2.5-456 shows the distribution of the percent compaction for the in-place tests. The lowest value recorded was 89.1% and the highest value was 107.6%.

There were a total of 7 in-place tests for the Baffle Dike that did not meet the 90% degree of compaction requirement. This represents approximately 0.3% of all of the in-place tests for the Baffle Dike. Their values ranged from 89.1% to 89.9%. Four of the eight tests had a value of 89.9%. Two of the failing tests were taken at the same location as another test (a nuclear test and a baloon test). The second test did meet the 90% compaction requirement in each case. These 7 tests represent isolated areas within the dike and have in-place compaction densities only slightly less than the minimum requirement. Therefore, they will not be detrimental to the integrity of the Baffle Dike.

There were a total of 11 in-place tests for the Baffle Dike which did not meet the placement moisture criteria. This represents approximately 0.5% of all the tests. Of these tests, nine had in-place moisture content that were only 0.1% above the specified limit. One had a moisture content 0.2% above the limit and the other was 0.5% above the limit. The test that was 0.5% above the limit also had another test at the same location and it was within the specified limits. These tests represent only isolated areas and are not considered to be detrimental to the integrity of the Baffle Dike.

Due to the highly preconsolidated nature of the underlying clays, such as the Illinoian till and the small additional load due to the embankment itself, no measurable consolidation or settlement is anticipated in the subgrade. No appreciable settlement is expected in the embankments themselves due to the small heights of the embankments.

2.5.6.4.1.2 <u>Slope Protection</u>

A soil cement mixture was placed on the baffle dike and the submerged dam for slope protection. For the baffle dike, the fill embankment was built to elevation 673 feet. The remaining 3 feet of the dike was constructed using the soil cement. It was placed in maximum 8-inch loose lifts and compacted using a smooth wheel (drum) compactor. The compacted material was then tested to verify that the specifications were met. Each succeeding lift was smaller in width to provide a stair-step effect to dissipate any wave action.

For the submerged dam, the soil cement was placed in maximum 8 inch loose lifts and compacted to a total 2 foot thick layer. It was placed parallel to the slopes which provided a relatively smooth surface for the outside of the embankment. Immediately downstream of the submerged dam, an area 56 feet wide at approximately elevation 670 feet received a 2-foot layered system of soil cement to prevent the possibility of undermining the dam. Also, the soil cement slope protection was placed and compacted on the north and south abutments of the submerged dam to protect them from scouring. No slope protection was provided for the baffle dike abutment which had a final slope of 4:1 (horizontal to vertical) above elevation 676 feet.

The soil cement was a mixture of soil aggregate, cement and water mixed to proper proportions at a central batching plant. The material was then transported and placed using dump trucks. The material was spread on a moist surface by either a pavement spreading machine or bulldozer. The in-place density requirements were a minimum dry density of 95% of the ASTM D-558 method with a moisture content within 1% above the optimum moisture content and 2% below the optimum. The test frequency was one test per 10,000 ft^2 per lift.

Statistical analyses of the in-place density tests for the soil cement slope protection for the UHS Dam and Baffle Dike were performed to verify the compaction requirements. The average dry density, the average moisture content, and the average percent compaction of the soil cement were determined using the results of the nuclear and sand cone tests performed. These averages were computed for the 1.) Baffle Dike, 2.) Slopes and crest of the Submerged Dam, and 3.) Submerged Dam including the abutments and downstream flat area.

A total of 126 in-place density tests were analyzed for the Baffle Dike. Figure 2.5-457 shows the distribution for the dry density test results. The average dry density was 130.1 PCF with the lowest value being 125.8 PCF and the highest value being 137.9 PCF. Figure 2.5-458 shows the distribution for the in-place moisture content. The average moisture content was 9.4% with the lowest value being 7.5% and the highest value being 11.1%. Figure 2.5-459 shows the distribution for the percent compaction for the in-place tests. The average percent compaction was 98.1% with the lowest value being 94.3% and the highest 104.4%.

There were 2 tests that did not meet the 95% compaction requirement, which were averaged with 2 other adjacent in-place tests. The average of each of these tests was greater than the 95% compaction requirement.

A total of 119 tests were performed on the crest, upstream slope, and downstream slope of the UHS Submerged Dam. Figure 2.5-460 shows the distribution for the results of the dry density tests. The average dry density was 126.8 PCF with the lowest value being 122.8 PCF and the highest value being 131.0 PCF. Figure 2.5-461 shows the distribution for the moisture content test results. The average moisture content was 10.7% with the lowest value being 9.3% and the highest value being 12.3%. Figure 2.5-462 shows the distribution for the percent compaction for the tests. The average percent compaction was 97.0% with the lowest value being 94.9% and the highest value being 99.9%.

There was only 1 in-place test that did not meet the 95% compaction requirement. This test was averaged with two additional adjacent tests. This average was greater than 95% compaction and was considered acceptable.

A total of 205 in-place density tests were analyzed when the tests performed on the dam abutments and downstream area were included in the summary. Figure 2.5-463 shows the distribution for the dry density test results. The average dry density was 126.7 PCF with the lowest value being 102.4 PCF and the highest being 132.4 PCF. Figure 2.5-464 shows the distribution for the moisture content test results. The average moisture content was 10.6% with the lowest value being 8.7% and the highest value being 12.3%. Figure 2.5-465 shows the distribution for the percent compaction for the test results. The average percent compaction was 96.5% with the lowest value being 77.5% and the highest value being 100.2%.

A total of 11 in-place tests did not meet the 95% compaction requirement. Three of these tests were averaged with two additional adjacent tests. The averages of these tests were greater than 95% compaction and were considered acceptable. The remaining 8 failing tests were performed on the downstream flat area immediately downstream of the submerged dam. These tests were taken on the middle layer of three placed in this area and were documented in a non-conformance report. This area was considered acceptable based on the fact that the layer is confined between two acceptable layers, appeared to be hard and solid on the surface, and would not be detrimental to the integrity of the submerged dam.

A total of 4 in-place tests performed on the soil cement did not meet the specified moisture content limits. All four were in the Baffle Dike area. Three of these were in the same area (one initial failure and two retests). The average moisture content of these three tests was 0.3% below the specified limits. The one other failing test value was averaged with two retests. The resulting average of these was within the specified limits. The percent compaction for all of these tests was acceptable. These failing tests represent only minor areas and they will not be detrimental to the integrity of the Baffle Dike.

Representative bag samples of the soil aggregate for the soil cement were obtained from the stockpile for every 4000 yd^3 of material used. A grain-size analysis (ASTM D-422) was performed on each sample to verify that the material met the specifications. Also, a five-point compaction test (ASTM D-588) was performed on representative samples for every 20,000 yd^3 of material placed. This test was used to develop a family of curves from which maximum dry densities and optimum moisture contents could be determined.

A three-point compaction method was used for the field control of in-place density and moisture requirements. A representative sample of placed soil cement was obtained and three compaction tests were performed at three different moisture contents: one as placed, one above, and one below. The curve defined by these three points was compared to the family of curves and a maximum dry density and optimum moisture content was then determined. This test procedure was performed for every 4000 yd^3 of material placed.

2.5.6.4.2 Main Dam

The main dam is a homogeneous earth embankment constructed using Type A cohesive fill material. The top of the dam is at elevation 711.8 feet with the average ground elevation at 655 feet, thus giving an average effective height of the dam of approximately 56.8 feet. The dam is approximately 3000 feet in length, with side slopes of 3:1 (horizontal to vertical). Riprap material was placed as slope protection on portions of both the upstream and downstream faces of the dam. A typical section of the main dam is presented in Figure 2.4-14.

2.5.6.4.2.1 Subgrade and Embankment Materials

The foundation materials for the main dam consisted of the Illinoian till and the dense alluvial sands. A comprehensive subgrade testing program for the main dam was initiated as described in Subsection 2.5.6.3.2. One in-place density test was performed for every 20,000 ft^2 of exposed subgrade. Laboratory testing on subgrade samples taken at each test location included: grain-size analysis (ASTM D-422) and Atterberg limit (ASTM D-423 and D-424) on cohesive samples, and relative density (ASTM D-2049) for granular samples.

Fill placement commenced immediately following subgrade approval. The borrow material was obtained from the excavation for the auxiliary spillway on the east side of the dam. The material used in the embankment was placed in near horizontal lifts not exceeding 8 inches in loose thickness and was compacted by heavy sheepsfoot roller compaction equipment. Following the completion of each day's work, the top of the embankment and borrow areas were bladed smooth and crowned slightly to promote surface runoff in the event of rain.

A 2-foot thick blanket drain was constructed in the embankment on the downstream side of the dam to control seepage. This is discussed in Subsection 2.5.6.6.2.

The subgrade for the service spillway was established on the Illinoian till at approximately elevation 667 feet and at elevation 630 feet for the stilling basin. The subgrade testing program previously described for the dam was also followed for this area. Due to the overexcavation to obtain suitable subgrade on the upstream side of the service spillway, a maximum of 12 feet of Type A fill was placed and compacted to a minimum dry density of 102% of the Standard Proctor maximum dry density as determined by ASTM D-698.

This material was placed in near horizontal lifts in loose thicknesses not exceeding 6 inches and was compacted using heavy sheepsfoot roller compaction equipment. The placement moisture content was between the optimum water content and 4% below the optimum.

A comprehensive laboratory and field testing program was established as an integral part of the Quality Control program at the site. Representative bag samples of the borrow material were taken for every 10,000 yd^3 of material placed to determine the physical characteristics of the material in the laboratory. Subsection 2.5.5.3.1 describes both the preliminary testing and field testing in conjunction with the construction of the dam.

The field testing program consisted of in-place density and moisture testing using either the Washington densometer method (ASTM D-2167) or the nuclear density method (ASTM D-2922). The in-place density and moisture requirements were a minimum of 95% of the maximum dry density determined by ASTM D-698 and a placed moisture content within 2.0% above the optimum moisture content and 4.0% below the optimum moisture content. The frequency of testing was one in-place density test per 10,000 ft^2 per lift.

Due to the highly preconsolidated nature of the underlying soils, such as the Illinoian till, no measurable consolidation or settlement is anticipated in the subgrade areas where the Illinoian till was exposed. In the areas where the dense alluvial sands formed the subgrade, any consolidation or settlement in these materials was expected to occur immediately and simultaneously with the construction of the embankment.

2.5.6.4.2.2 <u>Slope Protection</u>

Riprap material was placed on both the upstream and downstream faces of the main dam. On the upstream face of the dam, 18 inches of riprap was placed over two 9-inch layers of graded bedding materials. The riprap and bedding materials were placed from elevation 677 feet up to elevation 704 feet. From elevation 704 feet up to the top of the dam, only the riprap and one

18-inch to 9-inch bedding layer was placed. This design allowed for 13 feet of placed riprap beneath the normal pool elevation of 690 feet on the upstream face.

On the downstream face of the dam, riprap was placed from the toe of the dam up to elevation 670 feet. An 18-inch thick layer of the riprap was placed over two 9-inch layers of graded bedding material. Above elevation 670, a 4-inch layer of topsoil was placed and seeded for erosion control.

The riprap and related filters were designed following the recommendations of the U.S. Bureau of Reclamation (Reference 88) and the U.S. Army Corps of Engineers (Reference 89). The maximum size of the riprap was 30 inches while the average size was approximately 9 inches.

2.5.6.5 Slope Stability

2.5.6.5.1 Ultimate Heat Sink

The static and dynamic slope stability analyses for the submerged earthfill and baffle dike are discussed in Subsections 2.5.5.2.2, 2.5.5.2.3, and 2.5.5.2.4. The results of the laboratory testing are described in Subsection 2.5.5.3.2.

2.5.6.5.2 Main Dam

The slope stability analysis for the main dam is presented in Subsection 2.5.5.2.1. The results of the laboratory testing are presented in Subsection 2.5.5.3.1. The updated parameters used in the stability analysis were determined from a program of in-place testing involving the extraction of shelby tube samples from the embankment in the spring of 1977. These values were used to verify the design parameters determined by the initial laboratory tests. Triaxial testing was performed on these samples to determine the actual in situ strength obtained during construction. This program is discussed in Subsection 2.5.6.9.

2.5.6.6 Seepage Control

2.5.6.6.1 Ultimate Heat Sink

Field permeability tests were performed in the Illinoian till with the results being presented in Table 2.5-38. Also, laboratory permeability tests were performed on samples of the Illinoian till and are presented in Table 2.5-31. These tests indicate that the highest permeability for the subgrade materials for the submerged dam is 1.4×10^{-5} cm/sec.

Laboratory permeability tests were performed on remolded samples of the fill material. A coefficient of permeability of 2×10^{-8} cm/sec was calculated for the embankment material.

The dam and baffle dike were designed as submerged structures, to be operative only in the event that the cooling lake is lost. When in operation, the dam will only have a head of approximately 7 feet and seepage is expected to be minimal during the shutdown period. Due to the combination of low permeabilities, small head, and long seepage path, no significant amount of seepage is expected from the ultimate heat sink during the 30 day shutdown period when the heat sink is operational.

2.5.6.6.2 Main Dam

Laboratory and field permeability testing was performed on representative samples of in situ and remolded materials for the surrounding vicinity and the dam embankment. Coefficients of permeabilities for in situ material are presented in Tables 2.5-32 and 2.5-38 for the laboratory and field tests, respectively. The coefficients of permeability for remolded samples are presented in Table 2.5-33. The estimated coefficient of permeability for the embankment materials was 2 x 10^{-8} cm/sec.

A cutoff trench was constructed along the centerline of the dam to control the seepage beneath the dam. Because most of the dam was constructed on dense sands, the cutoff trench was excavated a minimum of 2 feet into the Illinoian till and had a minimum width of 40 feet. The cutoff trench was then filled with Type A fill material compacted in accordance with the specifications for the embankment fill.

As shown in Figure 2.4-14, a 2-foot thick sand blanket drain was placed from the downstream toe of the dam into the dam for a distance of 100 feet. This drain will prevent any seepage from emerging from above the immediate toe of the dam. A collector ditch was established approximately 20 feet from the toe of the dam to collect any seepage from the dam and control its flow away from the dam.

2.5.6.7 Diversion and Closure

2.5.6.7.1 Ultimate Heat Sink

The North Fork of Salt Creek was diverted through a channel excavated along the north side of the heat sink and across the dam centerline to allow the excavation and fill placement for the baffle dike and the submerged dam. A slurry trench was installed and a temporary dike was constructed along the south side of the diversion channel and around the west side of the submerged dam to prevent seepage into the excavations. For the section of the submerged dam north of the diversion channel, a slurry trench was constructed in a U-shape with the two open ends keying into the abutment. Seepage into the excavations was collected in a series of sumps and was pumped into the diversion channel as it became necessary.

After the baffle dike was completed and the north and south sections of the submerged dam were approximately 95% complete, a new diversion channel was excavated through the newly placed fill material in the north section of the dam. Earth cofferdams were constructed immediately upstream and downstream of the submerged dam where the original diversion channel flowed across the centerline of the dam. The new diversion channel allowed the flow in the North Fork to continue while the center section of the dam could be dewatered by pumping, excavated to suitable subgrade, and fill placed to tie in the two existing fill sections of the dam. After the completion of this section and the placement of the soil cement slope protection on the baffle dike and dam, a temporary cofferdam was constructed across the new diversion channel for the final closure of the dam and the impoundment of water in the heat sink. Natural ground in this area was above elevation 675 feet, being outside the limits of the heat sink, thus allowing this area to remain relatively dry while the heat sink was filling. Debris and overly wet material were removed from the new diversion channel prior to fill placement.

2.5.6.7.2 Main Dam

A slurry trench was used to prevent seepage from entering into the excavations for the main dam. The trench was U-shaped for each section of the dam on either side of Salt Creek and the open ends keyed into the abutments. The excavation for the cutoff trench along the dam centerline was performed first to allow the alluvial sands to drain to permit the excavation to the suitable subgrade sand in the dry. The seepage was collected in a sump from where it was pumped back into Salt Creek.

After both sections of the embankment were above approximately elevation 670, two earth cofferdams were constructed immediately upstream and downstream of the dam section in the original creek bed. The creek flow was then diverted over the existing fill in a channel excavated approximately 300 feet west of the original creek channel. This final section of the dam was dewatered by pumping, excavated to suitable subgrade, and fill was placed to tie into each of the other embankment sections. The fill was placed only up to approximately elevation 655 feet. At this time, the creek was diverted back to its original course over the newly placed fill in a new channel.

When both sections of the embankment were completed to approximately elevation 700 feet, the creek flow was stopped and fill placement of the closure section began. Debris and overly wet material was removed from the closure section prior to fill placement.

2.5.6.8 Performance Monitoring

2.5.6.8.1 Ultimate Heat Sink Monitoring Program

The ultimate heat sink (UHS) monitoring program consists of a visual inspection of the UHS shoreline (including the abutments of the UHS submerged dam) to detect scour or erosion around the UHS, a sediment survey of the UHS and immediate area upstream, a hydrographic survey of the UHS submerged dam, and a physical inspection of the submerged dam, the shutdown service water (SSW) outlet structure, and intake screenhouse. The sedimentation monitoring program for the ultimate heat sink is also discussed in Subsection 2.4.11.6.

The monitoring program shall be performed on an annual basis, except for sedimentation accumulation monitoring which shall be done periodically as required to effectively maintain the required UHS Volume. Sedimentation monitoring shall be scheduled based upon as-left results of dredging, past experience with accumulation rates, recent trends, and latest projections. Annual monitoring shall recommence when approximately 60% to 70% of allowable silt by volume is accumulated, or if sedimentation is approaching elevation 672 feet (msl) at more than two adjacent sections. Additional inspections shall be performed if a major flood, drought, or earthquake occurs. A major flood shall be defined as the 100-year or greater flood. A major drought shall be defined as the 100-year or greater drought for which the lake level is at elevation 682.3 feet. A major earthquake shall be defined as the operating basis earthquake (OBE) having a horizontal peak ground acceleration of 0.10g or larger at the site.

2.5.6.8.1.1 Monitoring Requirements

2.5.6.8.1.1.1 Inspection of UHS Shoreline

The UHS shoreline shall be observed from both the land and water (by boat) to determine if scour, erosion, or slope instability has occurred. Scour and erosion shall be defined as any area where the surface vegetation has been disturbed in an area greater than 100 square feet. Slope instability shall be defined as any area where cracks or dislodged trees are noticeable for a distance of 10 feet along the slope. When applicable, photographs and detailed sketches of any questionable areas shall be documented in the inspection report for comparison of conditions from past and future inspections.

The abutments of the UHS submerged dam at and above the water line shall be inspected from a boat for scour and erosion beneath and around the soil cement slope protection of the dam. Photographs and detailed sketches, when made, shall be documented in the inspection report.

2.5.6.8.1.1.2 Monitoring of UHS Submerged Dam

A permanent horizontal and vertical control system will be established on shore as shown on Figure 2.5-485 to provide a fixed guidance for the dam survey.

Annually, a fathometer will be used to measure and record continuous elevations along the crest of the submerged dam. Continuous elevation readings will also be taken for transverse sections of the submerged dam at intervals of 250 feet, measured from the permanent control system, alternating annually between 225 feet, 475 feet, 725 feet, etc., and 100 feet, 350 feet, 600 feet, etc. (Figure 2.5-485). Readings for transverse sections at Stations 3+50, 11+00, and 18+50 will be taken during each survey for a continuous record and comparison of the UHS dam slopes.

Whenever the fathometer readings indicate that an area of the crest of the submerged dam is 0.5 feet below the elevation of the crest as established in the 1982 survey, a resurvey will be performed. If the results of the resurvey concur with the first survey, an evaluation of the capacity of the UHS will be made to ensure that the UHS still has its 30-day minimum capacity. The minimum UHS volume, and the plant response to insufficient UHS volume, is given in the Technical Specifications.

All fathometer readings will be taken within 5 feet of the designated area and will be accurate to ±0.1 feet.

Horizontal and vertical control for the annual monitoring of the movement of the UHS submerged dam is accomplished in accordance with Figure 2.5-485 and with the use of a transit, a level, an EDM (Electronic Distance Measuring), a boat equipped with a recording fathometer. Lines of traverse for the boat are established and marked by buoys in the water. Control is maintained from preset stations on shore for the centerline of the submerged dam and the appropriate cross sections. With the use of the EMD and the fathometer, a continuous strip chart recording the elevation of the top of the dam at centerline is produced to give a profile for the entire dam. Cross sections taken at 250-foot intervals in a similar manner are developed by the fathometer.

From the data collected to date, it is evident that no measurable movement has taken place. Due to the submerged dams very flat cross section (1 vertical to 5 horizontal slopes), its 2-foot thick soil-cement facing and its load equilibrium, no movement is expected. Based on the above, it is known that the submerged dam is intact and is capable of performing its safety function.

The UHS dam is to be monitored for movement annually as described above except that cross sections will be taken between those originally taken, every other year, to more completely envelop the structure. Three cross sections, Station 3 + 50, 11 + 00' and 18 + 50, will be monitored every year to provide another form of comparison for movement. Figure 2.5-485 shows all of the cross sections. All of the above survey will be performed by licensed surveyors or licensed professional engineers experienced in this type of work (Q&R 241.22).

2.5.6.8.1.1.3 Sedimentation Monitoring of the UHS and Upstream Area

The initial survey of the UHS was performed in 1977 by using aerial photography. A topographic map at a scale of 1 inch equal to 100 feet with 2-foot contour intervals was prepared. From this map the initial volume of the UHS was computed.

Permanent horizontal and vertical control points have been established around the perimeter of the UHS to provide the base for the sedimentation monitoring program for the UHS. The 30 designated monitoring locations define nine North/South cross sections of the UHS as shown on Figure 2.4-31 which will be located from these control points.

A Raytheon Model DE719B survey fathometer, or equivalent, will be used to measure the bottom elevations of the UHS at these designated cross sections. The amount of sedimentation will be calculated based on the results of these measurements. All fathometer readings will be taken with 5 feet of the designated locations and will be accurate to ±0.1 feet.

The sedimentation monitoring program will be performed periodically as described in Subsection 2.5.6.8.1.

Whenever it is determined that the volume of sediment deposit in the UHS has reached 218 acre-feet, a dredging program will be undertaken.

2.5.6.8.1.1.4 Visual Inspection of Underwater Structures

A diver will inspect the concrete structures (SSW outlet structure and the intake screenhouse) for silt or debris accumulation at the intake or discharge points. The concrete structures will also be inspected for concrete deterioration, structural cracking, foundation undermining resulting from scour, and movement along construction joints.

Whenever structural cracking, concrete deterioration of more than 0.5 inches in depth over an area larger than 5 square feet is noticed in any UHS concrete structure, or scour beneath a UHS structure of more than 1 foot for a distance of 5 feet is reported, an evaluation of its effect on the structure will be made and documented. If it is determined that the condition is detrimental to the safe operation of the UHS, thus affecting the shutdown service water system, the plant may go into safe shutdown as directed by the Technical Specifications until the situation is remedied.

Photographs or detailed sketches of significant conditions will be made to allow an evaluation by qualified engineers.

2.5.6.8.1.1.5 Qualification of Inspection and Evaluation Personnel

The inspection and evaluation work will be performed under the direction of qualified engineers by personnel qualified for the particular inspections. They shall be able to identify signs of distress (slope instability, erosion, and deterioration of slope protection and concrete structures) and provide recommendations for corrective actions.

2.5.6.8.1.2 Technical Evaluation

The results of the monitoring programs will be summarized in a report. The results will be compared with the initial and previous reported conditions. Abnormal hazardous conditions observed during the inspection shall be reported immediately to the consulting engineers and to the NRC.

The minimum UHS volume, and the plant response to insufficient UHS volume, is given in the Technical Specifications.

2.5.6.8.1.3 Content of Inspection Report

A technical report shall be prepared to present the results of each inspection. The original report shall be kept onsite for the life of the plant. The inspection report shall include the following:

1. Statements as to the reason for the survey, such as annual report, major flood, earthquake, etc.

2. Results of visual inspections.

3. Results of the sedimentation monitoring survey.

4. Results of the UHS submerged dam and SSW outlet and intake structures monitoring program.

5. Comparisons at recently performed profiles to the initial and previous profiles of the UHS.

6. Assessments of the causes of abnormal conditions, evaluation of the UHS integrity, and recommendations for additional investigations, remedial measures, or future inspections, where appropriate.

7. A log of abnormal conditions (floods, seismic events, extreme reservoir drawdown, etc...) since the previous inspection.

8. Name and license numbers of the surveyors, engineers, and other personnel that performed the field work and report input.

9. Description of the boat, equipment, lake and weather conditions during the surveys.

10. Summary of daily lake level readings recorded by the owner.

2.5.6.8.1.4 Quality Assurance Requirements

The quality assurance/control procedures, special process procedures, and documentation that apply to the work will outline the testing, inspection, etc., activities necessary for the accomplishment of the work and the assurance of its quality and requisite documentation.

2.5.6.8.2 Main Dam

An instrumentation plan was established for the main dam to monitor seepage, settlement, and horizontal movement.

Three sets of six hydrostatic pressure cells were installed in the dam to monitor the hydrostatic pressure at various elevations in the dam. Twelve open standpipe piezometers were installed downstream of the dam to depths of 50 feet to monitor groundwater levels in the Illinoian till beneath the dam and in the alluvium and fill adjacent to the dam (Figure 2.5-272 and Table 2.4-31). Slightly elevated hydrostatic pressures were observed in piezometers in the Illinoian till near the left abutment (Figures 2.4-48 through 2.4-50). A system of five relief wells (three presently working) were installed in October 1979 to reduce the pressures. Seven additional piezometers were installed near these relief wells to monitor their performance. Additional discussion of the observation wells is presented in Subsections 2.4.13.2 and 2.4.13.4.

Three cross-arm reference points were established along the dam centerline to monitor the settlement in the embankment itself. Also, a system of triangulation points and surface monuments were established to monitor any horizontal or vertical movement of the embankment.

2.5.6.9 Construction Notes

An independent testing program was established in the spring of 1977 to determine the depth and extent of frost penetration in the completed portions of the heat sink and main dam embankments. Shelby tube samples were taken at depths up to 4 feet. Testing for these samples included: Atterberg limit, dry density, moisture content, and consolidated-undrained triaxial compression tests. Conservative values for the effective strength parameters used in the slope stability analyses were determined from this testing as shown in Table 2.5-52.

From this testing program it was determined that below a depth of 18 inches, the properties of the compacted fill met the design requirements. Therefore, the upper 12 inches were removed and the remaining 6 inches were disced open and recompacted with the heavy sheepsfoot compaction equipment before fill operations resumed in 1977.

2.5.6.9.1 Ultimate Heat Sink

The overexcavation for the submerged dam on the downstream side was backfilled with Type C material up to approximately elevation 668.5 feet. This material was obtained from the same borrow areas as the Type A material used in the embankments and it met the specification requirements of the Type A material. It was placed at moisture contents between 6% above the optimum moisture content and 3% below the optimum moisture content and compacted to a minimum dry density of 85% of the maximum dry density as determined by the modified Proctor compaction test (ASTM D-1557). For a distance of 56 feet downstream of the toe of the submerged dam, 2 feet of soil cement slope protection was placed over this Type C material. This soil cement was placed to protect the toe of the dam from being undermined in the event the main dam was lost. Beyond the 56-foot distance, random fill material was placed up to existing grade at approximately elevation 673.5 feet. A typical section of the submerged dam is shown in Figure 2.4-24.

Soil cement slope protection was used instead of riprap in the heat sink area to protect the surfaces of baffle dike and the submerged dam. The soil cement material proved to be more economically feasible as well as having sources of suitable borrow material more accessible. Soil cement is discussed in Subsection 2.5.6.4.1.2.

The final locations of the north abutment for the submerged dam and the abutment for the baffle dike were changed from their preliminary locations. Additional borings as discussed in Subsection 2.5.4.3 were taken to establish more suitable soil conditions.

The alignment of the submerged dam was changed for the northern section of the dam. This realignment permitted the dam abutment to be keyed into unweathered Illinoian till up to elevation 675 feet. The new alignment also prevented encroachment of the abutment fill into the right-of-way for Illinois Highway 54. This realignment is shown in Figure 2.5-384. Geologic mapping for the abutment is discussed in the mapping report presented in Attachment C2.5.

Type A cohesive material was not obtained from the station excavation as originally planned. Borrow areas south of the heat sink were determined to be able to provide a more homogeneous and greater quantity of suitable borrow material. These areas are discussed in Subsection 2.5.6.4.1.1.

2.5.6.9.2 Main Dam

Although the major portion of the main dam foundation was established on the dense alluvial sand, it became necessary to remove some areas of the sand that were too loose to be disturbed during construction. In these areas, the sand was completely removed and the Illinoian till was established as the subgrade.

Unsuitable borrow material was spoiled in designated areas both upstream and downstream of the main dam. Also, waste material was spoiled adjacent to the upstream slope of the dam up to elevation 670 feet. This material adds more stability to the upstream slope of the dam as having the effect of flattening the final slope to more than 3:1 (horizontal to vertical).

2.5.6.10 Operational Notes

The main dam was closed and impoundment of water in Lake Clinton commenced on October 12, 1977. The heat sink dam was closed on October 15, 1977, and had water flowing over the crest on October 21, 1977. No adverse effects of sloughing of the slopes at either dam location have been recorded.

2.5.7 References for Section 2.5

1. P. B. King, "The Evolution of North America," 190 pp., Princeton University Press, Princeton, New Jersey, 1959.

2. H. B. Willman, et al., "Handbook of Illinois Stratigraphy," Bulletin 95, 261 pp., Illinois State Geological Survey, Urbana, Illinois, 1975.

3. H. Faul, "Ages of Rocks, Planets, and Stars," 109 pp., McGraw-Hill, New York, 1966.

4. J. Boellstorf, "Proposed Abandonment of pre-Illinoian Pleistocene Stage Names," Geol. Soc. America, North Central Section 12th Annual Meeting, Abstracts, p. 247, 1978.

5. H. B. Willman, and J. C. Frye, "Pleistocene Stratigraphy of Illinois," Bulletin 94, 204 pp., Illinois State Geological Survey, Urbana, Illinois, 1970.

6. K. Piskin and R. E. Bergstrom, "Glacial Drift in Illinois: Thickness and Character," Circular 490, 35 pp., Illinois State Geological Survey, Urbana, Illinois, 1975.

7. J. C. Frye, et al., "Glacial Tills of Northwestern Illinois," Circular 437, 45 pp., Illinois State Geological Survey, Urbana, Illinois, 1969.

8. G. S. Boulton, "Modern Arctic Glaciers As Depositional Models For Former Ice Sheets," Quarterly Journal of the Geological Society of London, Vol. 128, pp. 361-393, London, England, 1972.

9. W. B. Howe and J. W. Koenig, "The Stratigraphic Succession in Missouri," Second Series, Vol. XL, 185 pp., Missouri Geological Survey and Water Resources, Rolla, Missouri, 1961.

10. R. H. Shaver, et al., "Compendium of Rock-Unit Stratigraphy in Indiana." Bulletin 43., 229 pp., Indiana Department of Natural Resources, Geological Survey, Bloomington, Indiana, 1970.

11. H. L. James, Stratigraphic Commission Note 40, Subdivision of the Precambrian, an interim scheme to be used by the U.S. Geological Survey, Bull. Amer. Assoc. Petrol. Geologists, V. 56, pp. 1128-1133, 1972.

12. L. N. Stout and D. Hoffman, "An Introduction to Missouri's Geologic Environment," Educational Series No. 3, 40 pp., Missouri Geological Survey and Water Resources, Rolla, Missouri, 1973.

13. H. R. Schwalb, "Paleozoic Geology of the Jackson Purchase Region, Kentucky," Report of Investigations 10, 40 pp., Kentucky Geological Survey, Lexington, Kentucky, 1969.

14. E. Atherton, "Tectonic Development of the Eastern Interior Region of the United States," Background Materials for Symposium on Future Petroleum Potential of NPC Region 9, Illinois Petroleum 96, pp. 29-43, Illinois State Geological Survey, Urbana, Illinois, 1971.

15. A. J. Eardley, "Structural Geology of North America," 624 pp., Harper & Brothers, New York, 1951.

16. J. W. Koenig, "The Lincoln Fold in Northeastern Missouri," Report of Investigations No. 27, pp. 75-80, Missouri Geological Survey and Water Resources, Rolla, Missouri, September 1961.

17. M. H. McCracken, "Structural Features of Missouri," Report of Investigations No. 49, 99 pp., Missouri Geological Survey and Water Resources, Rolla, Missouri, 1971.

18. K. E. Clegg, "The La Salle Anticlinal Belt in Illinois," Guidebook Series No. 8, pp. 106-110, Illinois State Geological Survey, Urbana, Illinois, 1970.

19. D. H. Swann and A. H. Bell, "Habitat of Oil in the Illinois Basin," Bulletin of the American Association of Petroleum Geologists, Vol. 42, pp. 447-472, American Association of Petroleum Geologists, Tulsa, Oklahoma, 1958.

20. A. H. Bell, et al., "Deep Oil Possibilities of the Illinois Basin," Circular 368, 38 pp., Illinois State Geological Survey, Urbana, Illinois, 1964.

21. H. B. Willman and J. N. Payne, "Geology and Mineral Resources of the Marseilles, Ottawa, and Streator Quadrangles," Bulletin 66, 388 pp., Illinois State Geological Survey, Urbana, Illinois, 1942.

22. T. C. Buschbach, Illinois State Geological Survey, written communication, 1973. Unpublished Notes, Attachment D2.5, CPS-FSAR.

23. H. S. McQueen, N. S. Hinchey, and K. Aid, "The Lincoln Fold in Lincoln, Pike, and Ralls Counties, Northeastern Missouri," Report of Investigations No. 27, Missouri Geological Survey and Water Resources, Rolla, Missouri, pp. 81-85, 1961.

24. A. H. Bell, "Subsurface Structure of the Base of the Kinderhook-New Albany Shale in Central and Southern Illinois," Report of Investigations 92, 13 pp., Illinois State Geological Survey, Urbana, Illinois, 1943.

25. E. P. DuBois and R. Siever, "Structure of the Shoal Creek Limestone and Herrin (No. 6) Coal in Wayne County, Illinois," Report of Investigations 182, 7 pp., Illinois State Geological Survey, Urbana, Illinois, 1955.

26. W. W. Rubey, "Geology and Mineral Resources of the Hardin and Brussel Quadrangles," Professional Paper 218, 179 pp., U.S. Geological Survey, U.S. Government Printing Office, Washington, D.C., 1952.

27. W. L. Calvert, "Sub-Trenton Structure of Ohio, with Views, on Isopach Maps and Stratigraphic Sections as Basis for Structural Myths in Ohio, Illinois, New York, Pennsylvania, West Virginia, and Michigan," American Association of Petroleum Geologists Bulletin, Vol. 58, No. 6, pp. 957-972, American Association of Petroleum Geologists, Tulsa, Oklahoma, 1974.

28. D. R. Kolata and T. C. Buschbach, "Plum River Fault Zone of Northwestern Illinois," Circular 491, 20 pp., Illinois State Geological Survey, Urbana, Illinois, 1976.

29. G. A. Ludvigson et al., "A Field Guide to the Plum River Fault Zone in East-Central Iowa," in R. R. Anderson, editor, 42nd Annual Tri-State Geol. Field Conf. Guidebook, Geology of East-Central Iowa, pp. I-1 - I-49, 1978.

30. D. R. Kolata et al., "The Sandwich Fault Zone of Northern Illinois," Illinois State Geological Survey, Circular 505, 1978.

31. A. V. Heyl, et al., "The Geology of the Upper Mississippi Valley Zinc-Lead District," Professional Paper 309, 310 pp., U.S. Geological Survey, U.S. Government Printing Office, Washington, D.C., 1959.

32. S. E. Harris, Jr. and M. C. Parker, "Stratigraphy of the Osage Series in Southeastern Iowa," Report of Investigations 1, 52 pp., Iowa Geological Survey, Iowa City, Iowa, 1964.

33. R. L. Brownfield, "Structural History of the Centralia Area," Report of Investigations No. 172, 31 pp., Illinois State Geological Survey, Urbana, Illinois, 1954.

34. A. V. Heyl, "The 38th Parallel Lineament and its Relationship to Ore Deposits," Economic Geology, Vol. 67, pp. 879-894, 1972.

35. Sargent & Lundy, Supplemental Discussion Concerning the Limit of the Northern Extent of Large Intensity Earthquakes Similar to the New Madrid Events, May 23, 1975.

36. J. A. Harrison, "Subsurface Geology and Coal Resources of the Pennsylvanian System in White County, Illinois," Report of Investigations No. 153, 34 pp., Illinois State Geological Survey, Urbana, Illinois, 1951.

37. J. M. Weller, R. M. Grogan, and F. E. Tippie, "Geology of the Fluorspar Deposits of Illinois," Bulletin 76, 147 pp., Illinois State Geological Survey, Urbana, Illinois, 1952.

38. J. W. Baxter and G. A. Desborough, "Areal Geology of the Illinois Fluorspar District, Part 2 - Karbers Ridge and Rosiclare Quadrangles," Circular 385, 40 pp., Illinois, State Geological Survey, Urbana, Illinois 1965.

39. W. N. Melhorn and N. M. Smith, "The Mt. Carmel Fault and Related Structural Features in South-Central Indiana," Report of Progress No. 16, 29 pp., Indiana Geological Survey, Bloomington, Indiana, 1959.

40. T. A. Dawson, "Map of Indiana Showing Structure on Top of Trenton Limestone," Miscellaneous Map 17, Indiana Geological Survey, Bloomington, Indiana, 1971.

41. L. E. Becker, Head, Petroleum Section, Indiana Geological Survey, written communication to A. K. Yonk, Sargent & Lundy Senior Geologist, May 22, 1975. Attachment D2.5, CPS-FSAR.

42. H. H. Gray, Head Stratigrapher, Indiana Geological Survey, written communication to G. E. Heim, Sargent & Lundy Senior Geologist, October 15, 1974. Attachment D2.5, CPS-FSAR.

43. M. E. Ostrom, Director and State Geologist, Wisconsin Geological and Natural History Survey, written communication to G. E. Heim, Sargent & Lundy Geotechnical Division Head, April 8, 1975. Attachment D2.5, CPS-FSAR.

44. M. E. Ostrom, "Geology Field Trip - Southwestern Dane County," 10 pp., University of Wisconsin Geological and Natural History Survey, Madison, Wisconsin, 1971.

45. T. C. Buschbach and R. Ryan, "Ordovician Explosion Structure at Glasford, Illinois," Bulletin of the American Association of Petroleum Geologists, Vol. 47, No. 12, pp. 2015-2022, American Association of Petroleum Geologists, Tulsa, Oklahoma, 1963.

46. G. H. Emrich and R. E. Bergstrom, " Des Plaines Disturbance, Northeastern, Illinois," Geological Society of America Bulletin, Vol. 73, pp. 959-968, Geological Society of America, Boulder, Colorado, 1962.

47. D. A. Green, "Trenton Structure in Ohio, Indiana, and Northern Illinois," Bulletin of the American Association of Petroleum Geologists, Vol. 41, pp. 627-642, American Association of Petroleum Geologists, Tulsa, Oklahoma, 1957.

48. T. C. Buschbach, Coordinator, Nuclear Facilities Siting Studies, Illinois State Geological Survey, written communication to Dr. T. Kemmis, Sargent & Lundy Geologist, April 25, 1977. Attachment D2.5, CPS-FSAR.

49. L. D. McGinnis, "Crustal Tectonics and Precambrian Basement in Northeastern Illinois," report of Investigations 219, 29 pp., Illinois, State Geological Survey, Urbana, Illinois 1966.

50. T. C. Buschbach and G. E. Heim, "Preliminary Geologic Investigations of Rock Tunnel Sites for Flood and Pollution Control in the Greater Chicago Area," Environmental Geology Note 52, 35 pp., Illinois State Geological Survey, Urbana, Illinois, 1972.

51. F. T. Thwaites, "Map of the Buried Pre-Cambrian of Wisconsin," Wisconsin Geological and Natural History Survey, Madison, Wisconsin, 1957.

52. L. D. McGinnis, et al., "The Gravity Field and Tectonics of Illinois," Circular 494, 24 pp., Illinois State Geological Survey, Urbana, Illinois, 1976.

53. D. C. Bond, et al., "Possible Future Petroleum Potential of Region 9 - Illinois Basin, Cincinnati Arch, and Northern Mississippi Embayment," AAPG Memoir 15, Future Petroleum Provinces of the United States - Their Geology and Potential, (Ed. by I. H. Cram, et al.), Vol. 2, p. 1165-1218, American Association of Petroleum Geologists, Tulsa, Oklahoma, 1971.

54. P. C. Heigold, "An Aeromagnetic Survey of Southwestern Illinois," Circular 495, 28 pp., Illinois State Geological Survey, Urbana, Illinois, 1976.

55. R. W. Patenaude, "Results of Regional Aeromagnetic Surveys of Eastern Upper Michigan, Central Lower Michigan, and Southeastern Illinois," Department of Geology Research Report 64-2, 51 pp., University of Wisconsin Geophysical and Polar Research Center, Madison, Wisconsin, April 1964.

56. L. D. McGinnis and P. C. Heigold, "Regional Maps of Vertical Magnetic Intensity in Illinois," Circular 324, 12 pp., Illinois State Geological Survey, Urbana, Illinois, 1961.

57. K. E. Clegg, "Subsurface Geology and Coal Resources of the Pennsylvanian System in DeWitt, McLean, and Piatt Counties, Illinois," Circular 473, Illinois State Geological Survey, Urbana, Illinois, 25 pp., 1972.

58. P. C. Heigold, L. D. McGinnis, and R. H. Howard, "Geologic Significance of the Gravity Field in the Dewitt, McLean County Area, Illinois," Circular 369, 16 pp., Illinois State Geological Survey, Urbana, Illinois, 1964.

59. J. Van Den Berg and T. F. Lawry, "Petroleum Industry in Illinois, 1975" Illinois Petroleum 110, 126 pp., Illinois State Geological Survey, Urbana, Illinois, 1976.

60. T. C. Buschbach and D. C. Bond, "Underground Storage of Natural Gas in Illinois - 1973," Illinois Petroleum 101, 71 pp., Illinois State Geological Survey, Urbana, Illinois, 1974.

61. J. Docekal, "Earthquakes of the Stable Interior, with Emphasis on the Midcontinent," Ph.D. Dissertation, University of Nebraska, Lincoln, Vols. 1 and 2, 1970.

62. J.B. Hadley and J.F. Devine, "Seismotectonic Map of the Eastern United States," U.S. Geol. Survey. Misc. Field Studies, Map MF-620, 3 sheets, 1974.

63. A.J. Eardley, "Structural Geology of North America, Second Edition" Harper and Row, New York, 1962.

64. P.B. King, "The Tectonics of Middle North America," Hafner Publishing Company, New York, 1951.

65. M.L. Fuller, "The New Madrid Earthquake," Bulletin 494, U.S. Geol. Surv., 1912.

66. O.W. Nuttli, "The Mississippi Valley Earthquakes of 1811 and 1812, Intensities, Ground Motion, and Magnitudes," Seismol. Soc. Amer. Bull. 63, 1:227-248, 1973.

67. R.G. Stearns and C.W. Wilson "Relationship of Earthquakes and Geology in West Tennessee and Adjacent Areas," Tennessee Valley Authority, 1972.

68. L.D. McGinnis and C.P. Ervin, "Earthquakes and Block Tectonics in the Illinois Basin," Geology, 2:517-519, 1974.

69. Public Service Indiana, Marble Hill Preliminary Safety Analysis Report, Docket Nos. 50-546 and 50-547, 1975.

70. Stone & Webster, "Faulting in the Anna Ohio Region" Amendment 12, Appendix 2I PSAR for WUP, Koshkonong Station, 1976.

71. Union Electric Company, Callaway Preliminary Safety Analysis Report, Docket Nos. 50-488 and 50-486, 1976.

72. J.L. Coffman and C.A. von Hake, "Earthquake History of the United States," National Oceanic and Atmos. Adm., Boulder, Colo., Pub. 41-1 (revised edition through 1970), 1973.

73. N.S. Shaler, "Earthquakes of Western United States," Atlantic Monthly, 24(445): 549-559, 1869.

74. Sargent & Lundy, "Supplemental Discussion Concerning the Limit of the Northern Extent of Large Intensity Earthquakes Similar to the New Madrid Events," May 23, 1975.

75. W. Stauder and et al, "Seismic Characteristics of Southeast Missouri as Indicated by a Regional Telemetered Microearthquake Array," Seismol. Soc. Amer. Bull. 66, 6:1953-1964, December 1976.

76. F.L. Fox and C.T. Spiker, "Intensity Rating of the Attica (N.Y.) Earthquake of August 12, 1929. A Proposed Reclassification," Earthquake Notes, Vol. 48, 1-2:37-46, Jan. - June 1977.

77. Kansas Gas and Electric Company/Kansas City Power & Light Company, Wolf Creek Preliminary Safety Analysis Report, Docket No. 50-482, 1974.

78. M.D. Trifunac and A.G. Brady, "On the Correlation of Seismic Intensity Scales with the Peaks of Recorded Strong Ground Motion," Seismol. Soc. Amer. Bull. 65, 1:139-162, 1975.

79(a). H. A. Merz and C. A. Cornell, "Seismic Risk Analysis Based on a Quadratic Magnitude-Frequency Law," Bulletin of the Seismological Society of America, Vol. 63, No. 6, pp. 1999-2006, December 1973.

79(b). C. A. Cornell and H. A. Merz, "Seismic Risk Analysis of Boston," Journal of the Structural Division, ASCE, Vol. 101, No. ST10, pp. 2027-2043, October 1975.

80. W. D. McClain and 0. H. Myers, "Seismic History and Seismicity of the Southeastern Region of the United States," Report No. ORNL-4582, Oak Ridge National Laboratory, Oak Ridge, Tennessee, June 1970.

81. I. N. Gupta, "Attenuation of Intensities Based on Isoseismals of Earthquakes in Central United States" Earthquake Notes, Vol. 47, No. 3, pp. 13-20, July-September 1976.

82. J. R. Murphy and L. J. O'Brien, "Analysis of a Worldwide Strong Motion Data Sample to Develop an Improved Correlation Between Peak Acceleration, Seismic Intensity, and Other Physical Parameters," NUREG-0402, Computer Sciences Corporation (Report prepared for USNRC), January 1978.

83. L. Bjerrum, "Problems of Soil Mechanics and Construction on Soft Clays," Proceedings of 8th International Conference on Soil Mechanics and Foundation Engineering, Moscow, U.S.S.R., 1973.

84. "Standard Methods for the Examination of Water and Wastewater", 13th Edition, American Public Health Association, et al., Washington, D.C., 1971.

85. H. B. Seed and J. M. Idriss, "Simplified Procedure for Evaluating Soil Liquefaction Potential," Journal of Soil Mechanics and Foundations Division, ASCE, 97(SM9): pp. 1249-1273, 1971.

86. H. B. Seed and W. H. Peacock, "Test Procedures for Measuring Soil Liquefaction Characteristics," Journal of Soil Mechanics and Foundations Division, ASCE, 97 (SM8): pp 1099-1119, August 1971.

87. A. Casagrande, "The Determination of Preconsolidation Load and its Practical Significance," in Proceedings of First International Conference on Soil Mechanics and Foundation Engineering, Cambridge, Massachusetts, 1936.

88. "Design of Small Dams", U.S. Department of the Interior, Bureau of Reclamation, U.S. Government Printing Office, Washington, D.C., 1965.

89. "Engineering and Design of Earth- and Rock-Fill Dams: General Design and Construction Considerations," Engineer Manual EM 110-2-2300, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi, 1971.

90. J. S. Templeton and H. B. Willman, "Guidebook for the Sixteenth Annual Field Conference of the Tri-State Geological Society," Guidebook Series 2, Illinois State Geological Survey, Urbana, Illinois, 47 pp., 1952.

91. G. V. Cohee and C. W. Carter, "Structural Trends in the Illinois Basin," Circular 59, Illinois State Geological Survey, Urbana, Illinois, 4 pp., 1940.

92. K. E. Clegg, "Subsurface Geology and Coal Resources of the Pennsylvanian System in Clark and Edgar Counties, Illinois," Circular 380, Illinois State Geological Survey, Urbana, Illinois, 28 pp., 1965.

93. H. M. Bristol and R. Prescott, "Geology and Oil Production in the Tuscola Area, Illinois," Circular 424, Illinois State Geological Survey, Urbana, Illinois, 34 pp., 1968.

94. H. B. Seed and W. H. Peacock, "Applicability of Laboratory Test Procedures for Measuring Soil Liquefaction Characteristics Under Cyclic Loading," Report No. EERC 70-8, Earthquake Engineering Research Center, University of California, Berkeley, California, 1970.

95. H. B. Seed and R. V. Whitman, "Design of Earthquake Retaining Structures for Dynamic Loads, ASCE Specialty Conference on Lateral Stresses in the Ground and Design of Earth Retaining Structures," Cornell University, Ithaca, N.Y., 1970.

96. J. P. Gould, "Lateral Pressures on Rigid Permanent Structures," ASCE Specialty Conference on Lateral Earth Pressures, Cornell University, 1970.

97. G. F. Sowers et al., "The Residual Lateral Pressures Produced by Compacting Soils," Fourth Interior Conference, SM&FE, Volume II, London, 1957.

98. K. L. Lee and H. B. Seed, "Cyclic Stress Condition Causing Liquefaction of Sand," Journal of Soil Mechanics and Foundations Division, ASCE, (SMI), January 1967.

99. H. B. Seed, "Landslides during Earthquakes due to Soil Liquefaction," Journal of the Soil Mechanics and Foundations Division, 94(SM5), September 1968.

100. I. M. Idriss and Y. Lacroix, "Preliminary Safety Analysis Report," Appendix 2E, Shearon Harris Nuclear Power Plant, Wake County, North Carolina, Carolina Power and Light Company, 1973.

101. H. B. Seed and C. K. Chan, "Clay Strength under Earthquake Loading Conditions," Journal of Soil Mechanics and Foundations Division, ASCE, 92(SM2): 4723-4733, 1966.

102. L. D. McGinnis, P. C. Heigold, C. P. Ervin and M. Heidari, "The Gravity Field and Tectonics of Illinois," Illinois State Geological Survey Circular 494, 28 pp., 1976.

103. P. C. Heigold, "An Aeromagnetic Survey of Southwestern Illinois," Illinois State Geological Survey Circular 495, 28 pp., 1976.

104. U. S. Nuclear Regulatory Commission, Office of Nuclear Reactor Regulation, "Safety Evaluation Report Related to Construction of Marble Hill Plant," Docket Nos. 50-546 and 50-547, 1977.

105. H. B. Seed, K. L. Lee, and I. M. Idriss, "Analysis of Sheffield Dam Failure," Journal of Soil Mechanics and Foundations Division, ASCE, 95(SM6): 1453-1490, 1969.

106. F. H. Kulhawy, J. M. Duncan, and H. B. Seed, "Finite Element Analyses of Stresses and Movements in Embankments during Construction," College of Engineering, Office of Research Services, University of California, Berkeley, California, Report Number TE-69-4, 1969.

107. J. M. Roesset, "Fundamental of Soil Amplification, Seismic Design for Nuclear Power Plants," Edited by R. J. Hansen, The M.I.T. Press, Cambridge, Massachusetts, 1970.

108. H. B. Seed et al., "Analysis of the Slides in the San Fernando Dams During the Earthquake of February 9, 1971," Report No. EERC 73-2, Earthquake Engineering Research Center, University of California, Berkeley, 1973.

109. W. L. Shannon and D. E. Hiltz, "Submarine Landslide at Seward, The Great Alaska Earthquake of 1964."

110. R. L. Shiffman, R. V. Whitman, and J. C. Jordan, "Settlement Problem Oriented Computer Language," Journal of Soil Mechanics and Foundations Division, ASCE, March 1970.

111. McDonnell Douglas Automation Company, "SEPOL (User's Manual)," 1972.

112. K. Terzaghi and R. B. Peck, "Soil Mechanics in Engineering Practice," John Wiley & Sons, Inc., New York, 1968.

113. H. B. Seed and W. H. Peacock, "Applicability of Laboratory Test Procedures for Measuring Soil Liquefaction Characteristics under Cyclic Loading," Report No. EERC 70-8, Earthquake Engineering Research Center, University of California, Berkeley, 1970.

114. H. B. Seed, "Landslides During Earthquakes Due to Liquefaction," Journal of the Soil Mechanics and Foundations Division, ASCE, Volume 94, No. SM5, Proceeding Paper 6110, pp. 1053-1122, September 1968.

115. E. D'Appolonia, "Dynamic Loadings," Journal of Soil Mechanics and Foundations Division, ASCE, No. SM1, Volume 96, p. 61, January 1970.

116. Site Specific Response Spectra for Clinton Power Station - Unit 1 of Illinois Power Company, Revision 0, February 1982, by Weston Geophysical Corporation.

TABLE 2.5-1
SUMMARY OF FOLDS

NAME AND STATE	MEANS OF IDENTIFICATION*	MAJOR MOVEMENT**
Illinois		
Ashton Arch	B,S	Late Paleozoic (Reference 90)
Cap au Gres Faulted Flexure	S, B	Post-Middle Mississippian, Pre-Pennsylvanian (Reference 26)
Clay City Anticline	B	Pre-Pennsylvanian, Pennsylvanian, and/or Post-Pennsylvanian (Reference 25)
Downs Anticline	B	Mississippian through Pennsylvanian (Reference 57)
Dupo-Waterloo Anticline	B,S	Late Missisippian, Pre-Pennsylvanian (Reference 22)
DuQuoin Monocline	B	Pennsylvanian or earlier (Reference 22)
Illinois Basin	S, B, G	Early to Late Paleozoic (Reference 2)
Kankakee Arch	S, B, G	Ordovician to Pennsylvanian (Reference 15)
La Salle Anticlinal Belt	S, B, G	Post-Mississippian to Permian (Reference 22)
Lincoln Anticline	S,B	Late-Mississippian to Early Pennsylvanian (Reference 23)
Marshall Syncline	B	Late or Post-Pennsylvanian (Reference 92)
Matoon Anticline	B	Late Paleozoic (Reference 92)
Mississippi River Arch	S, B, G	Late Paleozoic (Reference 22)
Moorman Syncline	S, B	Post-Pennsylvanian (Reference 20)
Murdock Syncline	B	Late or Post-Pennsylvanian (Reference 92)
Pittsfield-Hadley Anticline	B	Post Pennsylvanian (Reference 22)
Salem and Loudon Anticlines	B	Pennsylvanian and Post-Pennsylvanian (Reference 22)
Sangamon Arch	B, G	Devonian to Early Mississippian (Reference 22)

TABLE 2.5-1 (Cont'd)

NAME AND STATE	MEANS OF IDENTIFICATION*	MAJOR MOVEMENT**
Illinois (Cont'd)		
Structures associated with the Plum River Fault Zone	S, B, G	Pennsylvanian, Post-Pennsylvanian, Pre-Middle Illinoian (Pleistocene) (Reference 28)
Tuscola Anticline	B	Pennsylvanian, and Post-Pennsylvanian (Reference 93)
Iowa		
Bentonsport	B	Mississippian (Reference 32)
Burlington	B	Mississippian (Reference 32)
Oquawka	B	Mississippian (Reference 32)
Skunk River	B	Mississippian (Reference 32)
Sperry	B	Mississippian (Reference 32)
Missouri		
Auxvasse Creek Anticline	S	Post-Pennsylvanian (Reference 17)
Browns Station Anticline	S	Late Mississippian or Pennsylvanian (Reference Clinton PSAR)
College Mound-Bucklin Anticline	S	Later Part or Post-Pennsylvanian (Reference 17)
Crystal City Anticline	S	Post-Mississipian (Reference Clinton PSAR)
Cuivre Anticline	S	Post-Mississippian (Reference Clinton PSAR)
Davis Creek Anticline	B	Post-Mississippian (Reference Clinton PSAR)
Eureka-House Springs Anticline	S, B	Post-Mississippian (Reference 17)
Farmington Anticline	S, B	No older than Devonian (Reference 17)
Kruegers Ford Anticline	S, B	Post-Ordovician (Reference Clinton PSAR)

TABLE 2.5-1 (Cont'd)

NAME AND STATE	MEANS OF IDENTIFICATION*	MAJOR MOVEMENT**
Missouri (Cont'd)		
Mexico Anticline	S, B	Late or Post-Pennsylvanian (Reference Clinton PSAR)
Mineola Structure	S	Pennsylvanian, Post-Pennsylvanian (Reference Clinton PSAR)
Ozark Uplift	S, B, G	Paleozoic, Mesozoic, Tertiary (Reference 17)
Pershing-Bay-Gerald Anticline	S	Post-Mississippian, Early Pennsylvanian (Reference Clinton PSAR)
Plattin Creek Anticline	S	Post-Mississippian (Reference Clinton PSAR)
Troy Brussels Syncline	S, B	Late Mississippian or Early Pennsylvanian (Reference 26)
Wisconsin		
Meekers Grove Anticline	B, S	Late Paleozoic (Reference 31)
Mineral Point Anticline	B, S	Late Paleozoic (Reference 31)
Wisconsin Arch	S, B, G	Early to Late Paleozoic (Reference 15)

*S = surface mapping, B = borehole, G = geophysical

**Due to the absence of sediments representing the interval from Pennsylvanian to Cretaceous or Pleistocene time, the age of final movement on these structures cannot be precisely dated. However, based on stratigraphic relationships and geologic history outside of the regional area, final movement on the structures is considered to have occurred prior to Pleistocene time, and possibly before the end of the Paleozoic.

TABLE 2.5-2
SUMMARY OF FAULTS

FAULT NAME	MEANS OF IDENTIFICATION*	FAULT DISPLACEMENT	LAST MOVEMENT
Cap au Gres Faulted Flexure	S, B	Maximum structural relief of 1200 feet (Reference 26)	Late Plicene-Pre-Pleistocene (Reference 22)
Centralia Fault	S (in mines), B	Downthrown up to 200 feet on west side (Reference 22)	Post-Pennsylvanian, Pre-Pleistocene (Reference 22)
Chicago Area Basement Fault** (Inferred)	G, B	Downthrown 900 feet on southwest side (Reference 49)	Pre-Middle Ordovician (Reference 49)
Chicago Area Minor Faults (Inferred)	G	North or south side of faults downthrown, 55 feet max. displacement (Reference 50)	Post-Ordovician or Post-Silurian, Pre-Pleistocene (Reference 50)
Chicago Area Minor Faults	S	Few inches to Few Feet (Reference 50)	Post-Silurian, Pre-Pleistocene (Reference 50)
Fortville Fault	B	Downthrown 60 feet on southeast side (Reference 41)	Post-Devonian, Pre-Pleistocene (Reference 42)
(Janesville Fault)***,+ (Iniferred)	B	North side downthrown (Reference 51)	Phanerozioc But, Pre-Pleistocene possible Pre-Cretaceous (Reference 43, 51)
(Madison Fault)***,+ (Inferred)	B	North side downthrown (Reference 51)	Phanerozoic but Pre-Pleistocene, possibly Pre-Cretaceous (References 43 and 51)
Mt. Carmel Fault	S, B	Downthrown in excess of 200 feet on west side (Reference 39)	Early Pennsylvanian Reference 39)

TABLE 2.5-2 (Cont'd)

FAULT NAME	MEANS OF IDENTIFICATION*	FAULT DISPLACEMENT	LAST MOVEMENT
Northeast Trending Faults South of the Rough Creek Fault Zone	S, B	Graben structures present, northwest or southeast walls of faults downthrown. Displacements are variable, may be up to 2000 feet. (Reference 38)	Tertiary, possibly Pre-Cretaceous (Reference 14)
Oglesby Fault+ (Inferred)	B	Downthrown 1200 feet on west side (Reference 47)	Pre-Cretaceous (Reference Clinton PSAR)
Plum River Fault Zone	S, B, G	Downthrown up to 400 feet on north side (Reference 28)	Post-Silurian, Pre-Middle Illinoian (Reference 28)
Rough Creek Fault Zone	S, B, G	North side downthrown; max. reported displacement of 3400 feet along the fault zone. (Reference 22)	Post-Pennsylvanian Pre-Late Cretaceous (Reference 22)
Royal Center Fault	B	Downthrown 100 feet on southeast side (Reference 41)	Post-Middle Devonian, Pre-Pleistocene (Reference 42)
Ste. Genevieve Fault Zone	S, B, G	Net displacement along the fault zone is down to the north and east; max. displacement greater than 1000 feet, possibly as much as 2000 feet (Reference 22)	Post-Pennsylvanian, Pre-Pleistocene (Reference 20, Reference 2)
Sandwich Fault Zone	S, B, G	Downthrown up to 800 feet on northeast side (Reference 20)	Post-Pennsylvanian Pre-Pleistocene (Reference 30)
Tuscola Fault+ (Inferred)	B	Downthrown 2000 feet on west side (Reference 47)	Pre-Cretaceous (Reference Clinton PSAR)
Wabash Valley Fault Zone	B, G, S	Graben structures present; northwest or southeast sides of faults downthrown; max. displacement of 400 feet (Reference 22)	Post-Pennsylvanian, Pre-Pleistocene (Reference 22)

TABLE 2.5-2 (Cont'd)

FAULT NAME	MEANS OF IDENTIFICATION*	FAULT DISPLACEMENT	LAST MOVEMENT
(Waukesha Fault)***,+,† Inferred)	S, B	Downthrown 1500+ feet on southeast side (Reference 51)	Phanerozoic but Pre-Pleistocene, possibly Pre-Cretaceous (References 43 and 51)
Wisconsin Minor Faults: Dane County	S, B	Downthrown up to 300 feet on the northwest side (Reference 44)	Post-Ordovician, Pre-Pleistocene, possibly Pre-Cretaceous (References 43 and 44)
Wisconsin Minor Faults: Waukesha †	S	Downthrown 27 feet on southeast side (Reference 43)	Post-Silurian Pre-Pleistocene, possibly Pre-Cretaceous (Reference 43) and Geological Map of the U.S., U.S. Geological Survey, (1974)

Notes:

* S = surface, B = borehole, G = geophysical

** No confirmation has been made for this postulated fault.

*** Names in parentheses were assigned by Dames & Moore.

+ Recent authorities doubt the existence of these faults, see References 43 and 48 which also appear in Attachment D2.5.

† For the present interpretation of these faults see Reference 43, which also appears in Attachment D2.5.

TABLE 2.5-3
CHEMICAL ANALYSIS ON THE NO. 7 AND NO. 8 COAL FROM BORING P-38
BY THE ILLINOIS STATE GEOLOGICAL SURVEY

SAMPLE	CONDITION*	MOISTURE (%)	VOLATILE MATTER (%)	FIXED CARBON (%)	ASH (%)	TOTAL SULFUR (%)	BTU/LB
No. 8 coal	1	12.5	32.2	35.9	19.3	4.2	9,600
	2	--	36.8	41.1	22.1	4.8	10,900
	3	--	47.2	52.8	--	6.1	14,000
	4	16.3	37.7	46.0	--	--	12,200
	5	--	45.1	54.9	--	--	14,600
No. 7 coal	1	9.6	35.4	35.8	19.3	8.2	9,800
	2	--	39.1	39.6	21.3	9.1	10,800
	3	--	49.7	50.3	--	11.5	13,800
	4	12.8	40.9	46.3	--	--	12,600
	5	--	46.9	53.1	--	--	14,400

* Type of analyses noted as follows:
1. as received at laboratory,
2. moisture free,
3. moisture and ash free,
4. moist mineral matter free,
5. dry mineral matter free.

TABLE 2.5-4
EARTHQUAKE EPICENTERS,
37° to 45° NORTH LATITUDE,
84° to 93° WEST LONGITUDE

	DATE	LOCATION	NORTH LATITUDE	WEST LONGITUDE	MAXIMUM INTENSITY (MM)	FELT AREA (mi²)	REFERENCES
1.	1795 Jan 8	Franklin County, IL	37.9	89.0	IV-V		1
2.	1804 Aug 24	Fort Dearborn, IL	42.0	87.8	VI	30,000	1, 2, 3, 5
3.	1818 April 11	St. Louis, MO	38.6	90.2	III-IV	7,500	1
4.	1819 Sept 2	Perry County, MO	37.7	89.7	IV	15,000	2
5.	1819 Sept 16	Randolph County, IL	38.1	89.8	IV	9,600	1
6.	1819 Sept 17	Randolph County, IL	38.1	89.8	III-IV		1
7.	1820 Nov 9	Cape Girardeau, MO	37.3	89.5	IV	5,000	2, 3
8.	1827 July 5	St. Louis, MO	38.6	90.2	IV-V		1
9.	1827 July 5	Grant County, KY	38.7	84.6	IV	15,000	1, 2
10.	1827 July 5	New Albany, IN	38.3	85.8		165,000	1, 2
11.	1827 July 6	Cincinnati, OH	39.1	84.5	IV		1
12.	1827 Aug 6	New Albany, IN	38.3	85.8	VI		1, 2, 3, 5
13.	1827 Aug 7	New Albany, IN	38.3	85.8	VI		1, 2, 3, 5
14.	1827 Aug 14	St. Louis, MO	38.6	90.2	III		1
15.	1838 June 9	St. Louis, MO	38.6	90.2	VI	300	1
16.	1843 Feb 16	St. Louis, MO	38.6	90.2	IV-V	100,000	1
17.	1845	Putnam County, OH	41.1	84.2	II		1
18.	1850 April 4	Louisville, KY	38.3	85.8	IV		1, 2, 4
19.	1854 Feb 28	Lexington, KY	38.1	84.5	VI		15

TABLE 2.5-4 (Cont'd)

	DATE	LOCATION	NORTH LATITUDE	WEST LONGITUDE	MAXIMUM INTENSITY (MM)	FELT AREA (mi^2)	REFERENCES
20.	1854 Feb 28	Garrard County, KY	37.6	84.5	IV	8,000	2
21.	1855 May 2	Cairo, IL	37.0	89.2	IV		2
22.	1855 May 3	Cairo, IL	37.0	89.2	III		2
23.	1857 Oct 8	St. Louis, MO	38.6	90.3	VI-VII	7,500	1, 3
24.	1860 Aug 7	Henderson, KY	37.8	87.6	V	30,000	2
25.	1869 Feb 20	Lexington, KY	38.1	84.5	III-IV		1
26.	1871 July 24	Cairo, IL	37.0	89.2	III		2
27.	1871 July 25	St. Clair County, IL	38.5	90.0	III	1,000	1
28.	1872 Feb 8	Cairo, IL	37.0	89.2	III		1
29.	1872 March 26	Paducah, KY	37.1	88.6	III		2
30.	1873 April 22	Dayton, OH	39.8	84.2	III-IV		1
31.	1874 July 9	Cairo, IL	37.0	89.2	III-IV		2
32.	1875 June 18	Champaign County, OH	40.2	84.0	VII	40,000	1,2,3,6,8,14
33.	1876 Jan 27	Adrian, MI	41.9	84.0	NOT RECORDED		1
34.	1876 June	Anna, OH	40.4	84.2	I-III		1,8,14,19
35.	1876 Sept 24	Wabash County, IL	38.5	87.9	VI		1
36.	1876 Sept 25	Knox County, IN	38.5	87.7	VI	60,000	1,2,3,6,7
37.	1876 Sept 26	Wabash County, IL	38.5	87.9	III		1
38.	1877 May 26	New Harmony, IN	38.1	87.9	III-IV		1
39.	1877 June 3	Stanford, KY	37.5	84.7	III		2
40.	1877 July 15	Carbondale, IL	37.7	89.2	III-IV	9,500	2

TABLE 2.5-4 (Cont'd)

	DATE	LOCATION	NORTH LATITUDE	WEST LONGITUDE	MAXIMUM INTENSITY (MM)	FELT AREA (mi^2)	REFERENCES
41.	1878 Jan 8	Cairo, IL	37.0	89.2	III-IV	3,000	2
42.	1878 Nov 19	Cairo, IL	37.0	89.2	III	3,000	2
43.	1881 April 20	Goshen, IN	41.6	85.8	IV		1, 2
44.	1881 May 27	LaSalle, IL	41.3	89.1	VI		1, 2
45.	1881 Aug 29	Hillsboro, OH	39.2	83.6	III		1, 2
46.	1882 Feb 9	Anna, OH	40.4	84.2	V	100	1, 2, 3, 8, 14
47.	1882 July 20	Randolph County, IL	38.0	90.0	V	30,000	1, 2
48.	1882 Sept 27	Macoupin County, IL	39.0	90.0	VI	25,000	1, 2, 3
49.	1882 Oct 14	Macoupin County, IL	39.0	90.0	V	8,000	1, 2
50.	1882 Oct 15	Macoupin County, IL	39.0	90.0	V	8,000	1, 2, 3
51.	1882 Oct 22	Greenville, IL	38.9	89.4	III		1
52.	1882 Nov 15	St. Louis, MO	38.6	90.2	III		1
53.	1883 Jan 10	Union County, IL	37.5	89.3	III		2
54.	1883 Jan 11	Cairo, IL	37.0	89.2	VI	80,000	3
55.	1883 Feb 4	Kalamazoo County, MI	42.3	85.6	VI	150,000	1, 2, 3
56.	1883 April 12	Cairo, IL	37.0	89.2	VI-VII		2
57.	1883 July 6	Cairo, IL	37.0	89.2	III		2
58.	1883 July 14	Wickliffe, KY	37.0	89.1	IV-V	10,000	2
59.	1883 Nov 14	St. Louis, MO	38.6	90.2	IV	1,200	1
60.	1883 Dec 28	Bloomington, IL	40.5	87.0	III		16
61.	1884 Feb 15	Washington County, MO	37.8	90.8	III		2

TABLE 2.5-4 (Cont'd)

	DATE	LOCATION	NORTH LATITUDE	WEST LONGITUDE	MAXIMUM INTENSITY (MM)	FELT AREA (mi^2)	REFERENCES
62.	1884 March 31	Preble County, OH	39.6	84.8	II		1
63.	1884 Sept 19	Allen County, OH	40.7	84.1	V-VI	125,000	1, 2, 8, 14
64.	1884 Dec 23	Anna, OH	40.4	84.2	III		1, 5, 14
65.	1885 Dec 26	Bloomington, IL	40.5	89.0	III		1
66.	1886 March 1	Butlerville, IN	39.0	85.5	III-IV		1,2
67.	1886 March 18	Cairo, IL	37.0	89.2	III-IV	3,000	2
68.	1886 Aug 13	Indianapolis, IN	39.8	86.2	III-IV		1
69.	1887 Feb 6	Vincennes, IN	38.7	87.4	VI	75,000	1, 2, 3, 6, 7
70.	1887 Aug 2	Cairo, IL	37.0	89.2	V	65,000	2
71.	1889 Sept	Anna, OH	40.4	84.2	III		1, 8, 14
72.	1891 July 26	Evansville, IN	38.0	87.6	VI		1, 2, 3, 6
73.	1891 Sept 26	Cairo, IL	37.0	89.2	V		2
74.	1892	Anna, OH	40.4	84.2	IV-VI		1,8,14,19
75.	1895 Oct 31	Near Charleston, MO	37.0	89.4	VIII	1,000,000	2, 3
76.	1895 Nov 2	Near Charleston, MO	37.0	89.4	III-IV		1
77.	1895 Nov 17	Near Charleston, MO	37.0	89.4	III-IV		1
78.	1896 March 15	Sidney, OH	40.3	84.2	IV		1, 8, 14
79.	1897 Oct 31	Niles, MI	41.8	86.3	NOT RECORDED		1
80.	1898 June 6	Richmond, KY	37.8	84.3	III		2
81.	1899 Feb 8	Chicago, IL	41.9	87.6	IV-V		1
82.	1899 Feb 9	Chicago, IL	41.9	87.6	IV-V		1

TABLE 2.5-4 (Cont'd)

	DATE	LOCATION	NORTH LATITUDE	WEST LONGITUDE	MAXIMUM INTENSITY (MM)	FELT AREA (mi²)	REFERENCES
83.	1899 April 29	Dubois County, IN	38.5	87.0	VI-VII	40,000	1, 2, 6
84.	1899 Oct 10	St. Joseph, MI	42.1	86.5	IV		1
85.	1899 Oct 12	Kenosha, WI	42.6	87.8	NOT RECORDED		1
86.	1902 Jan 24	Maplewood, MO	38.6	90.3	VI	40,000	1, 3
87.	1902 March 10	Hagerstown, IN	39.9	85.2	III-IV		1
88.	1903 Jan 1	Hagerstown, IN	39.9	85.2	II-III		1
89.	1903 Feb 8	St. Louis, MO	38.6	90.3	VI	40,000	1, 3
90.	1903 March 17	Hillsboro, IL	39.2	89.5	III-IV		1
91.	1903 Sept 20	Morgantown, IN	39.4	86.3	IV		1
92.	1903 Sept 21	Olney, IL	38.7	88.1	IV		1
93.	1903 Nov 3	Murphysboro, IL	37.8	89.3	III-IV		2
94.	1903 Nov 4	St. Louis, MO	38.6	90.3	VII	70,000	Callaway PSAR
95.	1903 Nov 20	Morgantown, IN	39.4	86.3			1
96.	1903 Dec 11	Effingham, IL	39.1	88.5	II		1
97.	1903 Dec 31	Fairmont, IL	41.6	88.1			1
98.	1905 March 13	Menominee, MI	45.0	87.7	V		1, 3
99.	1905 April 13	Keokuk, IA	40.4	91.6	V	5,000	1, 2, 3
100.	1905 Aug 22	Quincy, IL	39.9	91.4	II-III		1
101.	1906 Feb 23	Anabel, MO	39.7	92.4	III		1
102.	1906 March 6	Hannibal, MO	39.7	91.4	IV		1
103.	1906 April 22	Milwaukee, WI	43.0	87.9			1

TABLE 2.5-4 (Cont'd)

	DATE	LOCATION	NORTH LATITUDE	WEST LONGITUDE	MAXIMUM INTENSITY (MM)	FELT AREA (mi^2)	REFERENCES
104.	1906 April 24	Milwaukee, WI	43.0	87.9			1
105.	1906 May 8	Shelby County, IN	39.5	85.8	III-IV	600	1
106.	1906 May 9	Columbus, IN	39.2	85.9	IV		1, 2, 3
107.	1906 May 11	Petersburg, IN	38.5	87.3	V	1,200	1, 2, 3
108.	1906 May 19	Grand Rapids, MI	43.0	85.7			1
109.	1906 May 21	Flora, IL	38.7	88.5	V	560	1, 2, 3, 6
110.	1906 Aug 13	Greencastle, IN	39.6	86.9	IV		1
111.	1906 Sept 7	Owensville, IN	38.3	87.7	IV	500	1
112.	1906 Nov 23	Anabel, MO	39.7	92.4	III		1
113.	1907 Jan 10	Menominee, MI	45.1	87.6			1
114.	1907 Jan 29	Morgan County, IN	39.5	86.6	V		1, 2
115.	1907 Jan 30	Greenville, IL	38.9	89.4	V		1
116.	1907 July 4	Farmington, MO	37.8	90.4	IV-V	400	2, 3
117.	1907 Nov 20	Stephenson County, IL	42.3	89.8	IV	100	1, 2
118.	1907 Nov 28	Stephenson County, IL	42.3	89.8	IV	100	1, 2
119.	1907 Dec 10	St. Louis, MO	38.6	90.2	IV		1
120.	1908 Oct 27	Cairo, IL	37.0	89.2	V	5,000	2, 3
121.	1908 Dec 27	Ballard County, KY	37.0	89.0	IV	30,000	2
122.	1908 Dec 31	Ballard County, KY	37.0	89.0	III		1
123.	1909 May 26	South Beloit, IL	42.5	89.0	VII	170,000	1, 2, 3, 5
124.	1909 July 18	Mason County, IL	40.2	90.0	VII	35,000	1, 2, 3

TABLE 2.5-4 (Cont'd)

	DATE	LOCATION	NORTH LATITUDE	WEST LONGITUDE	MAXIMUM INTENSITY (MM)	FELT AREA (mi^2)	REFERENCES
125.	1909 Aug 16	Monroe County, IL	38.3	90.2	IV-V	18,000	1
126.	1909 Sept 22	Lawrence County, IN	38.7	86.5	V	4,000	1, 2, 3
127.	1909 Sept 27	Robinson, IL	39.0	87.7	VII	30,000	1, 2, 3, 6, 10
128.	1909 Sept 27	Vincennes, IN	38.7	87.5	V	4,000	1, 2, 3, 6, 10
129.	1909 Oct 22	Ironton, MO	37.6	90.6	IV		2
130.	1909 Oct 22	Sterling, IL	41.8	89.7	IV-V		1, 2
131.	1909 Oct 22	Near Scott, KY	38.9	84.5	IV or less		1
132.	1909 Oct 23	Scott County, MO	37.0	89.5	V	40,000	2, 3
133.	1909 Oct 23	Robinson, IL	39.0	87.7	V	14,000	1, 2, 5
134.	1911 Feb 28	St. Louis County, MO	38.7	90.3	IV		1
135.	1911 July 29	Chicago, IL	41.9	87.6	IV-V		1, 2
136.	1912 Jan 2	Kendall County, IL	41.5	88.5	VI	40,000	1, 3
137.	1912 Sept 25	Rockford, IL	42.3	89.1	III-IV		1, 2
138.	1913 Oct 16	Sterling, IL	41.8	89.7	III-IV	4,000	1, 2
139.	1913 Nov 11	Louisville, KY	38.3	85.8	IV		1
140.	1914 Oct 7	Madison, WI	43.1	89.4	IV		1
141.	1914	Anna, OH	40.4	84.2	III		1, 8, 14
142.	1915 Feb 5	Harrisburg, IL	37.7	88.5	IV	400	2
143.	1915 Feb 18	Mound City, IL	37.1	89.2	IV	350	2
144.	1915 April 15	Olney, IL	38.7	88.1	II-III	3,000	1
145.	1916 Jan 7	Worthington, IN	39.1	87.0	III	3,000	1

TABLE 2.5-4 (Cont'd)

	DATE	LOCATION	NORTH LATITUDE	WEST LONGITUDE	MAXIMUM INTENSITY (MM)	FELT AREA (mi^2)	REFERENCES
146.	1916 Feb 17	Near New Burnside, IL	37.6	88.8	III		2
147.	1916 May 31	Madison, WI	43.1	89.4	II		1
148.	1916 Aug 24	Cairo, IL	37.0	89.2	IV	4,000	2
149.	1916	Clarke County, IA	41.1	93.8	II-III		1
150.	1917 April 9	Jefferson County, MO	38.1	90.6	VI	200,000	1, 3
151.	1918 Feb 17	Cairo, IL	37.0	89.2	III	3,000	2
152.	1918 Feb 22	Shiawassee County, MI	42.9	84.2	IV		1
153.	1918 July 1	Hannibal, MO	39.7	91.4	IV		1
154.	1919 Feb 10	Henderson County, KY	37.8	87.5	III-IV	2,000	2
155.	1919 May 25	Knox County, IN	38.5	87.5	V	18,000	1, 2, 3, 6
156.	1920 April 30	Centralia, IL	38.5	89.1	IV	4,000	1
157.	1920 May 1	St. Louis County, MO	38.5	90.5	V	10,000	1, 3
158.	1921 Feb 27	Cairo, IL	37.0	89.2	III	3,000	2
159.	1921 March 14	Crawfordsville, IN	40.0	86.9	IV	25,000	1
160.	1921 March 31	Mount Vernon, IN	37.9	87.9	IV		2
161.	1921 Sept 8	Waterloo, IL	38.3	90.2	IV	4,000	1
162.	1921 Oct 1	Harrisburg, IL	37.7	88.5	IV	4,000	2
163.	1921 Oct 9	Waterloo, IL	38.3	90.2	III	3,000	1
164.	1922 Jan 10	Mount Vernon, IN	37.9	87.9	IV-V	9,500	2
165.	1922 March 22	Pope County, IL	37.3	88.6	V	25,000	3
166.	1922 March 22	Massac County, IL	37.3	88.9	V	60,000	2

TABLE 2.5-4 (Cont'd)

	DATE	LOCATION	NORTH LATITUDE	WEST LONGITUDE	MAXIMUM INTENSITY (MM)	FELT AREA (mi²)	REFERENCES
167.	1922 March 22	Massac County, IL	37.3	88.9	V		2
168.	1922 March 23	Ballard County, KY	37.0	88.9	V	20,000	2
169.	1922 April 10	Monmouth, IL	40.9	90.7	II		1
170.	1922 July 7	Fond du Lac, WI	43.8	88.5	V		1, 2
171.	1922 Nov 26	Eldorado, IL	37.8	88.4	VI-VII	50,000	2
172.	1923 March 8	Greenville, IL	38.9	89.4	III-IV	4,000	1
173.	1923 May 6	Cairo, IL	37.0	89.2	III-IV	4,000	2
174.	1923 Nov 9	Tallula, IL	40.0	89.9	IV-V	600	1, 2, 3
175.	1923 Nov 28	Calhoun, KY	37.5	87.3	III		2
176.	1923 Nov 29	Mississippi County, MO	37.0	89.2	IV		2
177.	1924 April 2	Paducah, KY	37.1	88.6	IV		2
178.	1925 Jan 26	Waterloo, IA	42.5	92.3	II	200	1
179.	1925 March 3	Evanston, IL	42.0	87.7	II-III		1
180.	1925 April 4	Cincinnati, OH	39.1	84.5	IV or less		1, 8, 14
181.	1925 April 26	Vanderburgh County, IN	38.0	87.5	V-VI	100,000	1, 2, 3
182.	1925 July 13	Edwardsville, IL	38.8	90.0	V		1
183.	1925 Sept 2	Henderson County, KY	37.8	87.6	V-VI	75,000	2, 3
184.	1925 Sept 20	Henderson County, KY	37.8	87.6	IV	9,500	2
185.	1925 Oct	Anna, OH	40.4	84.2	III		1, 8, 14
186.	1926 March 22	Harrisburg, IL	37.7	88.5	IV	4,000	2
187.	1926 Oct 3	Princeton, IN	38.4	87.6	III		1

TABLE 2.5-4 (Cont'd)

	DATE	LOCATION	NORTH LATITUDE	WEST LONGITUDE	MAXIMUM INTENSITY (MM)	FELT AREA (mi²)	REFERENCES
188.	1927 Jan 31	Jackson, MO	37.4	89.7	IV	4,000	2
189.	1928 Jan 23	Near Mt. Carroll, IL	42.0	90.0	IV	400	1, 2
190.	1928 March 17	St. Louis, MO	38.6	90.2	I		1
191.	1928 April 15	Near Cape Girardeau, MO	37.4	89.7	III-IV		1
192.	1928 Oct 27	Shelby County, OH	40.4	84.1	III	100	1, 8, 14
193.	1929 Feb 14	Near Princeton, IN	38.3	87.6	III-IV	1,000	1
194.	1929 Feb 26	Arcadia, MO	37.5	90.6	III-IV		2
195.	1929 March 8	Shelby County, OH	40.4	84.2	V	5,000	1, 2, 3, 6, 8, 14
196.	1930 Feb 25	Near Cairo, IL	37.0	89.5	III		2
197.	1930 May 28	Near Hannibal, MO	39.7	91.3	III		1
198.	1930 June 26	Near Lima, OH	40.5	84.0	III-IV		1, 8, 14
199.	1930 June 27	Near Lima, OH	40.5	84.0	IV		1, 8, 14
200.	1930 Aug 8	Near Hannibal, MO	39.6	91.4	III-IV		1
201.	1930 Aug 29	Near Blandville, KY	37.0	89.2	IV	4,000	2
202.	1930 Sept 20	Anna, OH	40.4	84.2	VI		1, 2, 3, 8, 11, 14
203.	1930 Sept 29	Sidney, OH	40.3	84.2	III		1, 8, 14
204.	1930 Sept 30	Anna, OH	40.3	84.3	VII		1, 2, 3, 8, 9, 14
205.	1930 Oct	Anna, OH	40.4	84.2	III-IV		1, 8, 14
206.	1930 Dec 23	Near St. Louis, MO	38.6	90.5	III-IV	1,000	1
207.	1931 Jan 5	Elliston, IN	39.0	86.9	V	500	1, 2, 3, 12
208.	1931 March 21	Sidney, OH	40.3	84.2	III		1, 8, 14

TABLE 2.5-4 (Cont'd)

	DATE	LOCATION	NORTH LATITUDE	WEST LONGITUDE	MAXIMUM INTENSITY (MM)	FELT AREA (mi²)	REFERENCES
209.	1931 March 31	Shelby County, OH	40.4	84.1	III		1
210.	1931 June 10	Malinta, OH	41.3	84.0	V	1,500	1, 8, 14
211.	1931 Sept 20	Anna, OH	40.4	84.2	VII	45,400	1, 2, 3, 8,11, 12, 14
212.	1931 Oct 8	Anna, OH	40.4	84.2	III		1, 8, 14
213.	1931 Oct 18	Madison, WI	43.1	89.4	II-III		1
214.	1931 Dec 17	St. Louis, MO	38.6	90.2	II		1
215.	1931 Dec 31	Petersburg, IN	38.5	87.3	NOT RECORDED		1
216.	1934 Nov 12	Rock Island, IL	41.5	90.5	V-VI	5,000	1, 3
217.	1935 Jan 5	Moline, IL	41.5	90.6	III-IV	200	1, 2
218.	1935 Jan 30	Harrison County, MO	40.5	94.0	III		1
219.	1935 Feb 26	Burlington, IA	40.8	91.2	II-III		1
220.	1935 Oct 29	Pike County, IL	39.6	90.8	NOT RECORDED		1
221.	1936 Oct 8	Butler County, OH	39.3	84.4	III	700	1, 8, 14
222.	1936 Dec 25	Cincinnati, OH	39.1	84.5	III		1
223.	1937 March 2	Anna, OH	40.4	84.2	VI-VII	70,000	1, 2, 6, 8, 9, 12, 14
224.	1937 March 3	Anna, OH	40.4	84.2	IV		1, 2, 8, 11, 14
225.	1937 March 3	Anna, OH	40.4	84.2	III	200	1, 8, 14
226.	1937 March 8	Anna, OH	40.4	84.2	VII-VIII	150,000	1, 2, 3, 6, 12, 14

TABLE 2.5-4 (Cont'd)

	DATE	LOCATION	NORTH LATITUDE	WEST LONGITUDE	MAXIMUM INTENSITY (MM)	FELT AREA (mi^2)	REFERENCES
227.	1937 April 23	Anna, OH	40.4	84.2	III	200	1, 8, 14
228.	1937 April 27	Anna, OH	40.4	84.2	III	200	1, 8, 14
229.	1937 May 2	Anna, OH	40.4	84.2	IV		1
230.	1937 June 29	Peoria, IL	40.7	89.6	II		1
231.	1937 Aug 5	Near St. Louis, MO	38.5	90.2	II-III		1
232.	1937 Aug 5	Granite City, IL	38.7	90.2	II		1
233.	1937 Oct 16	Cincinnati, OH	39.1	84.5	II-III		1
234.	1937 Nov 17	Near Centralia, IL	38.6	89.1	V	8,000	1, 2, 3, 6, 12
235.	1938 Feb 12	Porter County, IN	41.6	87.0	IV-V	6,500	1, 2
237.	1939 March 18	Near Jackson Center, OH	40.4	84.0	III	500	1, 8, 14
238.	1939 June 17	Anna, OH	40.4	84.2	IV	400	1, 8, 14
239.	1939 July 9	Anna, OH	40.4	84.2	II		1, 8, 14
240.	1939 Nov 23	Monroe County, IL	38.2	90.1	V-VI	150,000	1, 3
241.	1939 Nov 24	Davenport, IA	41.6	90.6	II-III		1, 2
242.	1940 Jan 8	Louisville, KY	38.3	85.8	II-III		1
243.	1940 May 27	Louisville, KY	38.3	85.8	III		1, 2
244.	1940 May 31	Paducah, KY	37.1	88.6	IV-V	1,000	2
245.	1940 Nov 23	Monroe County, IL	38.2	90.1	VI	150,000	1
246.	1940 Dec 28	Near Evansville, IN	37.9	87.4	III	700	2
247.	1941 Oct 4	St. Louis, MO	38.6	90.2	I		1
248.	1941 Oct 21	Cairo, IL	37.0	89.2	III-IV	1,200	2

TABLE 2.5-4 (Cont'd)

	DATE	LOCATION	NORTH LATITUDE	WEST LONGITUDE	MAXIMUM INTENSITY (MM)	FELT AREA (mi²)	REFERENCES
249.	1941 Nov 15	Waterloo, IL	38.3	90.2	III		1
250.	1942 Jan	Winfield, MO	39.0	90.7	III		1
251.	1942 Jan 14	St. Louis, MO	38.6	90.2	UNKNOWN	600	1
252.	1942 Jan 29	St. Louis, MO	38.6	90.2	UNKNOWN		1
253.	1942 Jan 30	St. Louis, MO	38.6	90.2	UNKNOWN		1
254.	1942 March 1	Kewanee, IL	41.2	89.9	IV-V	3,700	1, 2
255.	1942 March 29	Harrisburg, IL	37.7	88.5	III-IV	200	2
256.	1942 Aug 31	Cairo, IL	37.0	89.2	III-IV		2
257.	1942 Nov 17	East St. Louis, IL	38.6	90.2	III-IV	200	1
258.	1942 Dec 27	Maplewood, MO	38.6	90.3	II		1
259.	1943 April 13	Louisville, KY	38.3	85.8	IV		1
260.	1943 April 18	Waterloo, IL	38.3	90.2	I		1
261.	1943 May 20	West Alton, MO	38.9	90.2	I		1
262.	1943 May 24	West Alton, MO	38.9	90.2	I		1
263.	1943 June 8	Webster Groves, MO	38.6	90.4	III-IV		1
264.	1943 June 15	House Springs, MO	38.4	90.6	I		1
265.	1943 June 18	House Springs, MO	38.4	90.6	I		1
266.	1943 Sept 14	Near St. Louis, MO	38.7	90.3	I		1
267.	1944 Jan 7	Near Jackson, MO	37.5	89.7	III-IV	900	1
268.	1944 March 16	Elgin, IL	42.0	88.3	II		1
269.	1944 Sept 25	St. Louis, MO	38.6	90.2	IV	25,000	1

TABLE 2.5-4 (Cont'd)

	DATE	LOCATION	NORTH LATITUDE	WEST LONGITUDE	MAXIMUM INTENSITY (MM)	FELT AREA (mi^2)	REFERENCES
270.	1944 Nov 13	Anna, OH	40.4	84.2	III-IV	18,000	1, 4, 18
271.	1945 Jan 15	Farmington, MO	37.8	90.4	IV	700	2
272.	1945 March 27	St. Louis, MO	38.6	90.2	II-III		1
273.	1945 May 21	Near St. Louis, MO	38.7	90.2	III-IV		1
274.	1945 Sept 23	Cairo, IL	37.0	89.2	III-IV		2
275.	1945 Nov 13	Cairo, IL	37.0	89.2	III-IV	11,000	2
276.	1946 Feb 24	Centralia, IL	38.5	89.1	IV-V	1,500	1, 2
277.	1946 Oct 7	Near Chloride, MO	37.5	90.6	IV-V	32,000	2
278.	1946 Nov 7	Washington County, MO	38.0	90.7	II-III		1
279.	1947 March 16	Kane County, IL	42.1	88.3	IV		1
280.	1947 May 6	Milwaukee, WI	43.0	87.9	V	3,000	1, 2
281.	1947 June 29	Near St. Louis, MO	38.4	90.2	VI	15,000	1, 3
282.	1947 Aug 9	Branch County, MI	42.0	85.0	VI	70,000	1, 2, 3
283.	1948 Jan 5	Centralia, IL	38.5	89.1	IV-V	300	1, 13
284.	1948 Jan 15	Madison County, WI	43.2	89.7	IV-V		1
285.	1948 April 20	Iowa City, IA	41.7	91.5	III-IV		1
286.	1949 June 8	Ste. Genevieve, MO	38.0	90.1	III	300	1
287.	1949 Aug 11	Clayton, MO	38.7	90.3	II		1
288.	1949 Aug 26	Defiance, MO	38.6	90.8	II-III		1
289.	1950 Feb 8	Lebanon, MO	37.7	92.7	V		1
290.	1950 April 20	Dayton, OH	39.8	84.2	IV		1, 8, 14

TABLE 2.5-4 (Cont'd)

	DATE	LOCATION	NORTH LATITUDE	WEST LONGITUDE	MAXIMUM INTENSITY (MM)	FELT AREA (mi²)	REFERENCES
291.	1951 Sept 19	Near Florissant, MO	38.9	90.2	III-IV	1,200	1
292.	1952 Jan 7	Champaign County, IL	40.3	88.3	II-III		1
293.	1953 May 6	Cairo, IL	37.0	89.2	III		2
294.	1953 May 15	Cairo, IL	37.0	89.2	III		2
295.	1953 Sept 11	Near Roxana, IL	38.6	90.1	VI	6,000	1, 3
296.	1953 Dec 30	Centralia, IL	38.5	89.1	IV	1,200	1
297.	1954 Aug 9	Petersburg, IN	39.5	87.3	IV-V		1, 2
298.	1955 April 9	Near Sparta, IL	38.1	89.8	VI	20,000	1, 3
299.	1955 May 29	Ewing, IL	38.1	88.9	III-IV		1
300.	1956 Jan 27	Anna, OH	40.4	84.2	V	2,000	1, 2, 8, 14
301.	1956 March 13	Fulton County, IL	40.5	90.2	IV	2,000	1
302.	1956 July 18	Oostburg, WI	43.6	87.8	IV		1
303.	1956 Oct 13	Near Milwaukee, WI	42.8	87.9	IV		1
304.	1956 Nov 25	Wayne County, MO	37.1	90.6	VI	21,500	2, 3
305.	1957 Jan 8	Waupun, WI	43.6	88.7	III-IV		1
306.	1957 March 26	Paducah, KY	37.1	88.6	V	300	2, 3
307.	1958 Jan 27	Ballard County, KY	37.0	89.0	V	300	3
308.	1958 Nov 7	Wabash County, IL	38.4	87.9	VI	33,300	1, 2, 3, 9
310.	1962 June 26	Saline County, IL	37.7	88.5	V-VI	17,500	2, 3
311.	1963 Aug 2	McCracken County, KY	37.0	88.8	IV-V	2,600	2, 3
312.	1965 March 6	Crawford County, MO	37.8	91.2	VI-VII		2

TABLE 2.5-4 (Cont'd)

	DATE	LOCATION	NORTH LATITUDE	WEST LONGITUDE	MAXIMUM INTENSITY (MM)	FELT AREA (mi^2)	REFERENCES
313.	1965 Aug 14	Pulaski County, IL	37.1	89.2	VI-VII	400	2, 3
314.	1965 Aug 15	Cape Girardeau County, MO	37.4	89.5	VI		3
315.	1965 Oct 20	Washington County, MO	37.8	91.1	VI	160,000*	1, 2, 3
316.	1967 Feb 2	Lansing, MI	42.7	84.5	IV		1
317.	1967 July 21	Madison County, MO	37.5	90.4	VI		2, 3
318.	1967 Aug 5	Jefferson County, MO	38.3	90.6	II		1
319.	1968 Nov 9	Hamilton County, IL	38.0	88.5	VII	580,000	1, 2, 3
320.	1968 Dec 11	Louisville, KY	38.3	85.8	V		1
321.	1969 Jan 20	Farmington, MO	37.8	90.4	III		1
322.	1971 Feb 12	Wabash County, IL	38.5	87.9	IV	1,300	1
323.	1972 June 9	St. Francois County, MO	37.7	90.4	IV		1
324.	1972 June 19	Ballard County, KY	37.0	89.1	IV		1
325.	1972 Sept 15	Lee County, IL	41.6	89.4	VI	40,000	1, 5
326.	1973 Jan 7	Hopkins County, KY	37.4	87.3	III		1
327.	1973 Jan 12	St. Francois County, MO	37.9	90.5	III		1
328.	1973 April 18	St. Clair County, IL	38.5	90.2	II-III		1
329.	1974 March 27	St. Louis, MO	38.5	90.1	II-III		17
330.	1974 April 3	Southern Illinois	38.6	88.1	VI		17
331.	1974 April 5	Eastern Missouri	38.6	90.9	IV or less		17
332.	1974 June 5	Kentucky	38.6	84.8	NOT RECORDED		17

* 245,000 mi^2 according to Reference 2.

TABLE 2.5-4 (Cont'd)

	DATE	LOCATION	NORTH LATITUDE	WEST LONGITUDE	MAXIMUM INTENSITY (MM)	FELT AREA (mi^2)	REFERENCES
333.	1974 June 5	Southern Illinois	38.6	89.9	V		17
334.	1974 Aug 22	Southern Illinois	38.2	89.7	V		17
335.	1976 April 8	Stinesville, IN	39.3	86.8	V		18

<u>NOTE</u>

Blank spaces in Table 2.5-4 indicate that data is not available.

TABLE 2.5-4 (Cont'd)

(References for Table 2.5-4)

1. M.M. Varma, "Seismicity of the Eastern Half of the United States," unpublished Ph.D. dissertation, Indiana University, Bloomington, 176 pp., 1975.

2. J. Docekal, "Earthquakes of the Stable Interior with Emphasis on the Midcontinent," Ph.D. dissertation, University of Nebraska, Lincoln, Vols. 1 and 2, 1970.

3. J.L. Coffman and C.A. von Hake (eds.), "Earthquake History of the United States," U.S. Dept. of Commerce, NOAA, Environmental Data Service, Boulder, Colo., Publication 41-1, revised edition through 1970, 1973.

4. B.C. Moneymaker, "Some Earthquakes in Tennessee and Adjacent States (1699 to 1850)," Tenn. Acad. Sci. Jour. 29, 3:224-233, 1954.

5. P.C. Heigold, "Notes on the Earthquake of September 15, 1972 in Northern Illinois," Illinois State Geol. Surv. Environmental Geol. Notes, No. 59, 1972.

6. R.R. Heinrich, "A Contribution to the Seismic History of Missouri," Seismol. Soc. Amer Bull. 31, 3:187-224, 1941.

7. B.C. Moneymaker, Tennessee Valley Authority, unpublished report, 1964.

8. E.A. Bradley and T.J. Bennett, "Earthquake History of Ohio," Seismol. Soc. Amer. Bull. 55, 4:745-752, 1965.

9. O.W. Nuttli, "State-of-the-Art for Assessing Earthquake Hazards in the United States," U.S. Army Engineers Waterways Experiment Station, Report 1, Design Earthquakes for the Central United States, 1973.

10. B.C. Moneymaker, "Earthquakes in Tennessee and Nearby Sections of Neighboring States (1901-1925)," Tenn-Acad. Sci. Jour. 32, 2:91-105, 1957.

11. R.R. Heinrich, "The Mississippi Valley Earthquake of June 20, 1947," Seismol. Soc. Amer. Bull. 40:7-19, 1950.

12. U.S. Coast and Geodetic Survey, "United States Earthquakes, 1920-1935 and United States Earthquakes, 1936-1940, "U.S. Department of Commerce, Environmental Science Services Administration, National Earthquake Information Center, 1968 reissue and 1969 reissue, respectively.

13. B.C. Moneymaker, "Earthquakes in Tennessee and Nearby Sections of Neighboring States (1926-1950)," Tenn. Acad. Sci. Jour. 33, 3:224-239, 1958.

14. E.F. Pawlowicz, "Earthquake Statistics for Ohio," Ohio Jour. Sci. 75, 2:111, March 1975.

15. B.C. Moneymaker, "Earthquakes of Kentucky," unpublished, Tenn. Valley Authority, undated.

TABLE 2.5-4 (Cont'd)

16. C.G. Rockwood, "Notes on American Earthquakes," Amer. Jour. Sci., Vol. 21, No. 13, 3rd series, 1984.

17. U.S. Geological Survey, "Preliminary Determination of Epicenters," USGS monthly publication.

18. U.S. Geological Survey, Earthquake Information, Bulletin, USGS, bimonthly publication.

19. M. C. Hansen "Earthquakes in Ohio," Ohio Division Geological Survey, Education Leaflet No. 9 (1975).

TABLE 2.5-5
EARTHQUAKES OCCURRING OVER 200 MILES FROM THE SITE FELT AT THE CLINTON SITE

DATE	MAXIMUM MODIFIED MERCALLI INTENSITY	LOCALITY	EPICENTER LOCATION (degrees) N. LAT.	EPICENTER LOCATION (degrees) W. LONG.	FELT AREA (mi²)	DISTANCE FROM SITE (mi)
1811						
December 16	XI	Northeastern Arkansas Gulf Coast Tectonic Province	35.5	90.5	2,000,000	350
1812	X-XI					
January 23		New Madrid, Missouri Gulf Coast Tectonic Province	36.6	89.5	2,000,000	285
1812	XI-XII					
February 7		New Madrid, Missouri Gulf Coast Tectonic Province	36.6	89.5	2,000,000	285
1886	X					
August 31		Charleston, South Carolina Atlantic Coast Tectonic Province	32.9	80.0	2,000,000	700
1895	VIII					
October 31		Charleston, Missouri Gulf Coast Tectonic Province	37.0	89.4	1,000,000	215
1937	VII-VIII					
March 8		Anna, Ohio Central Stable Region	40.4	84.2	150,000	245

TABLE 2.5-6
DIRECT SHEAR TEST DATA
D BORINGS

BORING NUMBER	ELEVATION (ft-in.)	SOIL TYPE*	GEOLOGIC UNIT	DRY DENSITY (lb/ft³)	FIELD MOISTURE CONTENT (percent)	NORMAL PRESSURE (lb/ft²)	YIELD STRENGTH (lb/ft²)	PEAK STRENGTH (lb/ft²)	
D-10	650.0	SM	Salt Creek Alluvium	107	17.3	1000	483	725	
D-10	650.0	SM	Salt Creek Alluvium	107	17.3	2000	705	1060	
D-10	647.0	SM.SP	Salt Creek Alluvium	85	34.1	3000	960	1440	
D-10	647.0	SM.SP	Salt Creek Alluvium	85	34.1	4000	985	1480	
D-10	642.0	SM.GP	Salt Creek Alluvium	131	8.3	3000	1025	1540	
D-10	642.0	SM.GP	Salt Creek Alluvium	131	8.3	6000	985	1480	
D-10	638.0	ML	Illinoian Glacial Till	140	8.8	2000	2880	4320	
D-10	587.0	ML	Illinoian Glacial Till	141	6.9	6000	4334	6500	Limit
D-11	619.8	ML	Illinoian Glacial Till	144	6.2	4000	---	6500	Limit
D-11	559.8	ML.CL	Illinoian Glacial Till	134	9.2	6000	---	6500	Limit
D-11	524.8	ML	Illinoian Glacial Till	125	12.6	6000	3770	5640	
D-24	649.0	SP	Salt Creek Alluvium	106	18.1	1000	420	630	
D-24	649.0	SP	Salt Creek Alluvium	106	18.1	2000	1035	1550	
D-24	646.0	SP	Salt Creek Alluvium	100	21.7	3000	380	570	
D-24	646.0	SP	Salt Creek Alluvium	100	21.7	4000	486	730	
D-24	636.0	SP	Salt Creek Alluvium	109	17.5	3000	466	700	
D-24	636.0	SP	Salt Creek Alluvium	109	17.5	6000	900	1350	
D-30	645.9	CL	Illinoian Glacial Till	138	7.4	2500	4000	6000	
D-34	619.8	ML	Illinoian Glacial Till	142	7.3	5000	---	6500	Limit

* See Figure 2.5-355 for definition of soil type symbols.

TABLE 2.5-7
DIRECT SHEAR TEST DATA
B BORINGS

BORING NUMBER	ELEVATION (ft-in.)	SOIL* TYPE	GEOLOGIC UNIT	FIELD DRY DENSITY (lb/ft³)	FIELD MOISTURE CONTENT (percent)	NORMAL PRESSURE (lb/ft²)	YIELD STRENGTH (lb/ft²)	PEAK STRENGTH (lb/ft²)
B-5	667.7	SM	Wisconsinan Glacial Till	131	10.1	2000	906	1360
B-5	667.7	SM	Wisconsinan Glacial Till	131	10.1	4000	1882	2830
B-8	661.8	SM	Salt Creek Alluvium	118	12.3	2000	1465	2200
B-8	661.8	SW	Salt Creek Alluvium	118	12.3	4000	2465	3700

* See Figure 2.5-355 for definition of soil type symbols.

TABLE 2.5-8
UNCONFINED COMPRESSION TEST DATA
REMOLDED SAMPLES**

BORING NUMBER	ELEVATION (feet)	SOIL TYPE*	GEOLOGIC UNIT	REMOLDED DATA DRY DENSITY (lb/ft^3)	REMOLDED DATA COMPACTION EFFORT* (percent)	REMOLDED DATA MOISTURE CONTENT (percent)	SHEAR STRENGTH (lb/ft^2)
S-5	714.0	CL	Wisconsinan Glacial Till	113	89	16.3	1542
S-5	714.0	CL	Wisconsinan Glacial Till	112	88	16.2	1420
S-5	714.0	CL	Wisconsinan Glacial Till	109	86	17.9	850
S-5	714.0	CL	Wisconsinan Glacial Till	108	85	18.3	800

* Modified AASHO (T-180).

** As compacted.

Table 2.5-9
UNCONFINED COMPRESSION TEST DATA
REMOLDED SAMPLES AS COMPACTED

BORING NUMBER	ELEVATION (ft-in.)	SOIL TYPE*	GEOLOGIC UNIT	REMOLDED DATA DRY DENSITY (lb/ft^3)	REMOLDED DATA COMPACTION EFFORT** (percent)	REMOLDED DATA MOISTURE CONTENT (percent)	SHEAR STRENGTH (lb/ft^2)
S-9	712.2 - 707.2	CL	Wisconsinan Glacial Till	122	102.1	13.4	2,960
S-9	712.2 - 707.2	CL	Wisconsinan Glacial Till	122	102.1	13.1	3,500
S-9	712.2 - 707.2	CL	Wisconsinan Glacial Till	120	100.4	13.6	700
S-10	702.6 - 697.6	CL	Wisconsinan Glacial Till	118	91.0	15.2	690
S-10	702.6 - 697.6	CL	Wisconsinan Glacial Till	122	94.2	14.2	1,020
S-10	702.6 - 697.6	CL	Wisconsinan Glacial Till	119	91.9	13.7	950
S-10	702.6 - 697.6	CL	Wisconsinan Glacial Till	124	95.8	11.6	2,390
S-10	702.6 - 697.6	CL	Wisconsinan Glacial Till	125	96.5	12.1	2,880
S-10	702.6 - 697.6	CL	Wisconsinan Glacial Till	120	92.6	13.9	870
S-10	702.6 - 697.6	CL	Wisconsinan Glacial Till	116	89.6	15.3	720
S-10	702.6 - 697.6	CL	Wisconsinan Glacial Till	117	90.3	15.5	520
S-10	702.6 - 697.6	CL	Wisconsinan Glacial Till	121	93.4	12.4	1,760
S-12	719.2 - 712.2	CL	Wisconsinan Glacial Till	117	95.9	14.8	1,560
S-12	719.2 - 712.2	CL	Wisconsinan Glacial Till	105	86.0	14.7	1,440

* See Figure 2.5-355 for the definition of soil type symbols.
** AASHO Test Designation T-180.

TABLE 2.5-9 (Cont'd)

BORING NUMBER	ELEVATION (ft-in.)	SOIL TYPE*	GEOLOGIC UNIT	REMOLDED DATA DRY DENSITY (lb/ft³)	REMOLDED DATA COMPACTION EFFORT** (percent)	REMOLDED DATA MOISTURE CONTENT (percent)	SHEAR STRENGTH (lb/ft²)
S-12	719.2 - 712.2	CL	Wisconsinan Glacial Till	116	95.0	15.5	1,400
S-14	705.2 - 700.2	CL	Wisconsinan Glacial Till	125	96.1	11.6	1,380
S-14	705.2 - 700.2	CL	Wisconsinan Glacial Till	118	90.7	14.2	585
S-14	705.2 - 700.2	CL	Wisconsinan Glacial Till	113	86.9	16.1	305
S-14	705.2 - 700.2	CL	Wisconsinan Glacial Till	114	87.7	12.7	1,320
S-14	705.2 - 700.2	CL	Wisconsinan Glacial Till	113	86.9	15.2	475
S-14	705.2 - 700.2	CL	Wisconsinan Glacial Till	121	93.1	13.1	1,220
S-14	705.2 - 700.2	CL	Wisconsinan Glacial Till	115	88.4	15.1	460
S-14	727.2 - 720.2	CL	Wisconsinan Glacial Till	103	86.6	14.5	1,520
S-14	727.2 - 720.2	CL	Wisconsinan Glacial Till	115	96.6	12.9	4,000

* See Figure 2.5-355 for the definition of soil type symbols.
** AASHO Test Designation T-180.

TABLE 2.5-10

CONSOLIDATED-UNDRAINED TRIAXIAL TEST DATA
WITH PORE PRESSURE MEASUREMENTS

BORING NUMBER	ELEVATION (ft-in.)	GEOLOGIC UNIT	CONSOLIDATION PRESSURE $\sigma_c' = \sigma_3'$ (lb/ft^2)	PEAK SHEAR STRENGTH $(\sigma_1'-\sigma_3')/2$ maximum (lb/ft^2)	STRESSES at $(\sigma_1'-\sigma_3')/2$ maximum			$(\sigma_1'+\sigma_3')/2$ (lb/ft^2)
					u	σ_1'	σ_3'	
D-8	631.7	Illinoian Glacial Till	3,000	19,592	-418	52,676	13,492	33,084
D-8	591.7	Illinoian Glacial Till	6,000	14,955	-331	43,446	13,536	28,491
D-48	709.3	Wisconsinan Glacial Till	2,000	1,009	187	3,831	1,813	2,822
D-48	704.3	Wisconsinan Glacial Till	4,000	6,434	-1,584	18,451	5,584	12,018
D-48	689.3	Wisconsinan Glacial Till	6,000	7,089	-158	20,335	6,158	13,247

TABLE 2.5-11
CONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST DATA
WITH PORE PRESSURE MEASUREMENTS

BORING NUMBER	ELEVATION (ft-in.)	GEOLOGIC UNIT	CONSOLIDATION PRESSURE $\sigma_c' = \sigma_3'$ (lb/ft ²)	PEAK SHEAR STRENGTH $\sigma_1'-\sigma_3'$ MAX./2 (lb/ft ²)	STRESSES AT $\sigma_1'-\sigma_3'/2$ MAX (lb/ft²)			
					u	$\sigma_{1'}$	$\sigma_{3'}$	$\sigma_1'+\sigma_3'/2$ (lb/ft ²)
P-38	648.5	Illinoian Glacial Till	6,480	19,646	*	45,773	6,480	26,127
P-38	572.9	Lacustrine Deposits	10,000	7,415	4,291	20,531	5,702	13,116
H-23	707.3	Wisconsinan Glacial Till	2,160	3,414	-403	9,391	2,563	5,977
H-23	692.3	Interglacial Soil	11,808	7,672	5,126	22,024	6,682	14,353
H-38	673.4	Illinoian Glacial Till	3,528	11,245	-504	26,522	4,032	15,277
H-3	645.1	Illinoian Glacial Till	2,016	4,611	-329	11,567	2,345	6,956
H-20	721.8	Wisconinan Glacial Till	1,440	9,052	-4,464	24,009	5,904	14,956

* Test performed on specimen at field moisture content; pore pressure measured negligible.

TABLE 2.5-11 (Cont'd)

BORING NUMBER	ELEVATION (ft-in.)	GEOLOGIC UNIT	CONSOLIDATION PRESSURE $\sigma_c' = \sigma_3'$ (lb/ft ²)	PEAK SHEAR STRENGTH $\sigma_1'-\sigma_3'$ MAX./2 (lb/ft²)	STRESSES AT $\sigma_1'-\sigma_3'/2$ MAX (lb/ft²) u	$\sigma_{1'}$	$\sigma_{3'}$	$\sigma_{1'}+\sigma_{3'}/2$ (lb/ft ²)
H-24	670.7	Salt Creek Alluvium	388	1,610	-1,224	4,832	1,612	3,222
H-38	712.9	Wisconinan Glacial Till	4,997	2,813	-245	10,868	5,242	8,055
H-38	687.9	Interglacial Soil	4,608	3,888	850	11,534	3,758	7,646
H-25	633.7	Illinoian Glacial Till	8,640	26,403	-3,312	64,758	11,952	38,355
H-13	673.6	Salt Creek Alluvium	389	1,785	-535	4,494	924	2,709
H-6	504.3	Illinoian Glacial Till	9,994	7,667	3,989	21,338	6,005	13,672
H-25	674.7	Salt Creek Alluvium	100	509	-115	1,235	215	725

TABLE 2.5-12
UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSURE TEST DATA
REMOLDED SAMPLES *

BORING NUMBER	ELEVATION (ft-in.)	SOIL TYPE**	GEOLOGIC UNIT	REMOLDED DATA: DRY DENSITY (lb/ft^3)	REMOLDED DATA: COMPACTION EFFORT*** (percent)	REMOLDED DATA: MOISTURE CONTENT (percent)	CONFINING PRESSURE (lb/ft^2)	SHEAR STRENGTH (lb/ft^2)
S-14	705.2 to 700.2	CL	Wisconsinan Glacial Till	116	89.2	14.9	2000	276
S-14	705.2 700.2	CL	Wisconsinan Glacial Till	119	91.5	13.7	4000	877
S-14	705.2 to 700.2	CL	Wisconsinan Glacial Till	116	89.2	14.9	6000	544

* As compacted.

** See Figure 2.5-355 for definition of soil type symbols.

*** AASHO Test Designation T-180.

TABLE 2.5-12 (Cont'd)

				REMOLDED DATA				
BORING NUMBER	ELEVATION (feet)	SOIL TYPE**	GEOLOGIC UNIT	DRY DENSITY (lb/ft³)	COMPACTION EFFORT*** (percent)	MOISTURE CONTENT (percent)	CONFINING PRESSURE (lb/ft²)	SHEAR STRENGTH (lb/ft²)
Combined sample from:								
P-30 P-32 P-33 P-35	727-717	CL	Wisconsinan Glacial Till	130.2	96.8	10.1	1500	4273
Combined sample from:								
P-30 P-33 P-35	713-698	CL	Wisconsinan Glacial Till	126.7	97.2	11.0	1500	2424
Combined sample from:								
P-37 P-39 P-42	730-720	CL	Wisconsinan Glacial Till	123.0	91.8	11.7	1500	1106
Combined sample from:								
P-30 P-32 to P-35 P-37 P-39 P-42	727-692	CL	Wisconsinan Glacial Till	129.2	--	9.9	1500	3674

TABLE 2.5-12 (Cont'd)

BORING NUMBER	ELEVATION (feet)	SOIL TYPE**	GEOLOGIC UNIT	REMOLDED DATA DRY DENSITY (lb/ft^3)	REMOLDED DATA COMPACTION EFFORT*** (percent)	REMOLDED DATA MOISTURE CONTENT (percent)	CONFINING PRESSURE (lb/ft^2)	SHEAR STRENGTH (lb/ft^2)
Combined sample from:								
P-30 P-33 P-35	713 to 698	CL	Wisconsinan Glacial Till	122.9	91.0	8.7	1728	1364 (Saturated)
Combined sample from:								
P-34 P-37 P-39 P-42	720 to 700	CL	Wisconsinan Glacial Till	129.8	97.0	10.3	1728	1631 (Saturated)

TABLE 2.5-13
CONSOLIDATED-UNDRAINED TRIAXIAL TEST DATA
WITH PORE PRESSURE MEASUREMENTS
REMOLDED SAMPLES*

BORING NUMBER	ELEVATION (ft-in.)	GEOLOGIC UNIT	REMOLDED DATA: MOISTURE CONTENT (percent)	REMOLDED DATA: DRY DENSITY (lb/ft^3)	REMOLDED DATA: COMPACTION EFFORT** (percent)	REMOLDED DATA: CONSOLIDATION PRESSURE $\sigma_c'=\sigma_3'$ (lb/ft^2)	PEAK SHEAR STRENGTH $\sigma_1'-\sigma_3'/2$ Max (lb/ft^2)	STRESSES AT $\sigma_1'-\sigma_3'/2$ Max (lb/ft^2): u	σ_1'	σ_3'	$\sigma_1'+\sigma_3'/2$ (lb/ft^2)
S-10	717.6-712.6	Wisconsinan Glacial Till	15.5	121	89.4	2,620	3,069	-43	8,801	2,663	5,732
S-10	717.6-712.6	Wisconsinan Glacial Till	13.4	119	87.9	4,075	3,504	1,396	9,688	2,679	6,184
S-10	717.6-712.6	Wisconsinan Glacial Till	12.6	123	90.8	7,257	5,942	1,958	17,183	5,299	11,241
S-10	702.6-697.6	Wisconsinan Glacial Till	14.0	118	91.1	2,000	1,801	518	5,014	1,412	3,212
S-10	702.6-697.6	Wisconsinan Glacial Till	13.5	120	92.6	4,000	2,170	2,260	6,069	1,728	3,899
S-10	702.6-697.6	Wisconsinan Glacial Till	13.8	121	93.4	6,000	4,523	10,166	13,280	4,234	8,757

* As compacted.

** AASHO Test Designation T-180.

TABLE 2.5-14
CONSOLIDATED-UNDRAINED TRIAXIAL TEST DATA
WITH PORE PRESSURE MEASUREMENTS
REMOLDED SAMPLES

			REMOLDED DATA				PEAK SHEAR STRENGTH	STRESSES AT $\sigma_1'-\sigma_3'/2$ Max (lb/ft^2)			
BORING NUMBER	ELEVATION (ft-in.)	GEOLOGIC UNIT	MOISTURE CONTENT (percent)	DRY DENSITY (lb/ft^3)	COMPACTION EFFORT** (percent)	CONSOLIDATION PRESSURE $\sigma_c'=\sigma_3'$ (lb/ft^2)	$\sigma_1'-\sigma_3'/2$ Max (lb/ft^2)	u	σ_1'	σ_3'	$\sigma_1'+\sigma_3'/2$ (lb/ft^2)
Combined sample from:											
P-30	713-6	Wisconsinan	8.5	123.6	94.8	2,000	2,983	+173	7,793	1,827	4,810
P-33	98	Glacial									
P-35		Till									
Combined sample from:											
P-37	730-7	Wisconsinan	12.8	123.1	91.6	8,000	4,909	+3,960	13,858	4,040	8,949
P-39	20	Glacial									
P-42		Till									

TABLE 2.5-15
CONSOLIDATED-UNDRAINED TRIAXIAL TEST DATA
WIITH PORE PRESSURE MEASUREMENT
REMOLDED SAMPLES

BORING NUMBER	ELEVATION (ft-in.)	GEOLOGIC UNIT	REMOLDED DATA: MOISTURE CONTENT (percent)	REMOLDED DATA: DRY DENSITY (lb/ft^3)	REMOLDED DATA: COMPACTION EFFORT** (percent)	CONSOLI8DATION PRESSURE $\sigma_c'=\sigma_3'$ (lb/ft^2)	PEAK SHEAR STRENGTH $\sigma_1'-\sigma_3'/2$ Max (lb/ft^2)	STRESSES AT $\sigma_1'-\sigma_3'/2$ Max (lb/ft^2): u	σ_1'	σ_3'	$\sigma_1'+\sigma_3'/2$ (lb/ft^2)
Structural fill borrow:		Salt Creek Alluvium	12.1[a]	118.8 [a]	67.0 [a]	1,008	7,235	-2,995	18,473	4,003	11,238
Combined bulk sample from:			- [a]	- [a]	- [a]	4,032	10,125	-2,174	26,456	6,003	16,331
G-18 G-19	663-654 673-663	Salt Creek Alluvium	12.4[b]	118.0 [b]	65.0 [b]	4,032	11,519	-2,059	29,128	6,091	17,610
G-20	657-647		- [b]	- [b]	- [b]	7,000	16,016	-2,505	41,537	9,505	25,521
			- [b]	- [b]	- [b]	9,936	18,238	-1,022	47,433	10,958	29,196
Structural fill borrow:		Salt Creek Alluvium	9.0[c]	120.2 [c]	70.5 [c]	876	10,832	-5,501	28,043	6,380	17,212
Combined bulk sample from:			- [c]	- [c]	- [c]	3,975	11,655	-3,370	30,678	7,368	19,023
G-18 G-19	663-654 673-663		- [c]	- [c]	- [c]	6,984	15,290	-2,808	40,375	9,794	25,084
G-20	657-647	Salt Creek Alluvium	8.4[d]	129.4 [d]	91.0 [d]	3,984	19,104	-5,501	47,698	9,490	28,594
			- [d]	- [d]	- [d]	5,688	19,102	-4,090	47,983	9,778	28,880
			- [d]	- [d]	- [d]	8,640	18,030	-3,470	48,170	12,110	30,140

TABLE 2.5-15 (Cont'd)

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* ASTM D2049-69 Relative Density of Cohesionless Soils.

[a] Mϕ/TX/CU/pp: Multiphase triaxial compression test, saturated, consolidated-undrained with pore pressure measurement; utilizing the same specimen initially molded at 12.1% moisture content and 118.8 lb/ft^3 dry density.

[b] Mϕ/TX/CU/pp, utilizing the same specimen initially molded at 12.4% moisture content and 118.0 lb/ft^3 dry density.

[c] Mϕ/TX/CU/pp, utilizing the same specimen initially molded at 9.0% moisture content and 120.2 lb/ft^3 dry density.

[d] Mϕ/TX/CU/pp, utilizing the same specimen initially molded at 8.4% moisture content and 129.4 lb/ft^3 dry density.

TABLE 2.5-16
MOISTURE AND DENSITY DATA
ULTIMATE HEAT SINK BORINGS

BORING NUMBER	ELEVATION (ft-in.)	SOIL TYPE	GEOLOGIC UNIT	DRY DENSITY (lb/ft³)	MOISTURE CONTENT (%)
H-1	667.7	ML	Salt Creek Alluvium	93	29.2
H-6	619.3	ML	Illinoian Glacial Till	136	8.6
H-6	579.3	CL	Illinoian Glacial Till	129	10.3
H-6	425.8	CH	Pre-Illinoian Lacustrine Deposit	120	15.2
H-14	635.3	ML	Illinoian Glacial Till	140	9.5
H-15	691.3	CL	Interglacial Zone	105	17.9*
H-17	669.6	ML	Salt Creek Alluvium	98	24.1
H-17	641.6	ML	Illinoian Glacial Till	139	8.7
H-20	731.8	ML	Wisconsinan Glacial Till	119	13.4
H-20	721.8	ML	Wisconsinan Glacial Till	130	9.5
H-20	716.8	ML	Wisconsinan Glacial Till	134	10.3*
H-20	706.8	ML	Wisconsinan Glacial Till	136	8.6
H-20	691.8	ML	Interglacial Soil	113	16.6
H-20	672.3	ML	Illinoian Glacial Till	135	10.5
H-22	540.8	ML	Pre-Illinoian Glacial Till	128	12.3
H-23	730.8	ML	Weathered Loess	109	12.8
H-23	707.3	ML	Wisconsinan Glacial Till	121	13.6

* Saturated

TABLE 2.5-16 (Cont’d)

BORING NUMBER	ELEVATION (ft-in.)	SOIL TYPE	GEOLOGIC UNIT	DRY DENSITY (lb/ft³)	MOISTURE CONTENT (%)
H-23	692.3	ML	Interglacial Soil	114	17.4
H-23	677.8	ML	Illinoian Glacial Till	138	9.1
H-30	638.5	ML	Illinoian Glacial Till	145	6.8
H-30	633.5	ML	Illinoian Glacial Till	141	7.4
H-31	667.1	ML	Illinoian Glacial Till	139	8.7
H-32	616.1	ML	Illinoian Glacial Till	134	8.4
H-32	575.1	CL	Lacustrine Deposit	116	14.6
H-36	622.7	ML	Illinoian Glacial Till	137	9.4
H-36	597.2	ML	Illinoian Glacial Till	139	7.7
H-37	689.9	ML	Interglacial Soil	122	6.5
H-38	712.9	ML	Wisconsinan Glacial Till	124	10.3
H-38	707.9	ML	Wisconsinan Glacial Till	122	12.4
H-38	687.9	ML	Inter-Glacial Soil	113	17.4
H-38	673.4	ML	Illinoian Glacial Till	135	8.6
H-40	672.6	ML	Salt Creek Alluvium	86	24.7

TABLE 2.5-17
UNCONSOLIDATED-UNDRAINED TRIAXIAL TEST DATA
ULTIMATE HEAT SINK BORINGS

BORING NUMBER	ELEVATION (ft-in.)	SOIL TYPE	GEOLOGIC UNIT	CONFINING PRESSURE (lb/ft²)	SHEAR STRENGTH (lb/ft²)	DRY DENSITY (lb/ft³)	MOISTURE CONTENT (%)
H-14	645.3	ML	Illinoian Glacial Till	1900	6902*	143	8.2*
H-16	652.8	ML	Illinoian Glacial Till	2002	1056	137	8.3
H-20	691.8	ML	Interglacial Soil	3197	3736*	115	17.9*
H-23	730.8	ML	Weathered Loess	346	610*	114	20.4*
H-23	697.3	ML	Interglacial Soil	2534	2974*	100	25.0*
H-30	658.0	ML	Illinoian Glacial Till	2693	7569	135	7.9
H-30	618.5	ML	Illinoian Glacial Till	5396	954	120	11.1
H-38	707.9	ML	Wisconsinan Glacial Till	3090	1238*	120	14.1*
H-38	692.9	CL	Interglacial Soil	2189	1626*	104	21.4*
H-23	662.8	ML	Illinoian Glacial Till	4750	1400	128	13.4
H-6	564.3	ML	Illinoian Glacial Till	7200	6840*	129	10.7*

* Saturated

TABLE 2.5-17 (Cont'd)

BORING NUMBER	ELEVATION (ft-in.)	SOIL TYPE	GEOLOGIC UNIT	CONFINING PRESSURE (lb/ft^2)	SHEAR STRENGTH (lb/ft^2)	DRY DENSITY (lb/ft^3)	MOISTURE CONTENT (%)
H-17	672.6	ML	Salt Creek Alluvium	288	850*	92	24.7*
H-14	668.3	SM	Salt Creek Alluvium	360	916*	108	19.1*
H-9	692.1	ML	Interglacial Soil	2600	1735*	108	20.5*
H-15	651.8	ML	Illinoian Glacial Till	4565	7745*	138	8.5*
H-22	554.8	ML	Pre-Illinoian Glacial Till	6984	2580*	127	13.0*
H-19	675.0	ML	Salt Creek Alluvium	200	396*	110	22.1*
H-15	696.3	ML	Interglacial Soil	4850	1350*	82.4	35.4*

* Saturated

TABLE 2.5-18
DYNAMIC TRIAXIAL COMPRESSION TEST DATA
STATION SITE BORINGS

BORING NUMBER	ELEVATION (ft-in.)	SOIL TYPE	FIELD MOISTURE CONTENT (percent)	DRY DENSITY (lb/ft³)	SINGLE AMPLITUDE SHEAR STRAIN (percent)	MODULUS OF RIGIDITY (lb/ft²)	DAMPING (percent)
P-14	714.3	ML	16.6	117	0.0141	1.25×10^6	17
					0.0264	1.07×10^6	16
					0.0521	0.85×10^6	16
(Wisconsinan Glacial Till)					0.1268	0.55×10^6	18
					0.2508	0.40×10^6	19
					0.5191	0.25×10^6	20
					1.2540	0.11×10^6	20
					1.9153	0.09×10^6	21
					2.5205	0.08×10^6	21
P-14	624.3	CL	8.3	139	0.0127	2.60×10^6	11
					0.0262	1.99×10^6	12
(Illinoian Glacial Till)					0.0508	1.53×10^6	15
					0.1272	0.91×10^6	19
					0.2502	0.66×10^6	21
					0.5077	0.46×10^6	22
					0.9921	0.35×10^6	21
					1.9841	0.24×10^6	19
					3.8697	0.33×10^6	20
P-15	692.3	ML	18.0	111	0.0144	1.18×10^6	14
					0.0277	0.97×10^6	12
(Interglacial Zone)					0.0531	0.84×10^6	11
					0.1037	0.61×10^6	12
					0.2529	0.39×10^6	15
					0.5059	0.26×10^6	18
					1.2647	0.15×10^6	21
					2.5170	0.09×10^6	25
					3.6941	0.06×10^6	29
					5.0841	0.05×10^6	32

TABLE 2.5-19
DYNAMIC TRIAXIAL COMPRESSION TEST DATA
ADDITIONAL STATION SITE BORINGS

BORING NUMBER	ELEVATION (ft-in.)	SOIL TYPE	FIELD MOISTURE CONTENT (percent)	DRY DENSITY (lb/ft³)	SINGLE AMPLITUDE SHEAR STRAIN (percent)	MODULUS OF RIGIDITY (lb/ft²)	DAMPING (percent)
P-32	657.4	ML	7.9	135	0.0063	2.45×10^6	11
(Illinoian Glacial Till)					0.0117	2.09×10^6	13
					0.0175	1.78×10^6	14
					0.0400	1.14×10^6	17
					0.0751	0.78×10^6	21
					0.1562	0.43×10^6	20
					0.2674	0.26×10^6	19
					0.5474	0.14×10^6	18
P-32	617.4	ML	9.6	133	0.0065	1.05×10^6	18
(Illinoian Glacial Till)					0.0167	0.76×10^6	15
					0.0283	0.59×10^6	17
					0.0575	0.40×10^6	18
					0.1133	0.27×10^6	18
					0.2196	0.17×10^6	21
					0.3798	0.12×10^6	20
					0.7596	0.08×10^6	20
					1.5248	0.05×10^6	19
P-32	547.4	ML	10.5	131	0.0242	2.15×10^6	
(Pre-Illinoian Glacial Till)					0.0653	1.21×10^6	
					0.1306	0.82×10^6	19
					0.2177	0.62×10^6	20
					0.3508	0.47×10^6	20
					0.9193	0.27×10^6	21
					1.8870	0.18×10^6	21
P-36	668.2	ML	13.0	132	0.0670	0.14×10^6	28
(Illinoian Glacial Till, low blow count material and test sample may also have been slightly disturbed)					0.0131	0.11×10^6	26
					0.0304	0.07×10^6	24
					0.0628	0.05×10^6	24
					0.1195	0.03×10^6	25
					0.2403	0.02×10^6	21
					0.4006	0.02×10^6	20

TABLE 2.5-19 (Cont'd)

BORING NUMBER	ELEVATION (ft-in.)	SOIL TYPE	FIELD MOISTURE CONTENT (percent)	DRY DENSITY (lb/ft³)	SINGLE AMPLITUDE SHEAR STRAIN (percent)	MODULUS OF RIGIDITY (lb/ft²)	DAMPING (percent)
P-36	578.7	ML	8.5	138	0.0059	1.98×10^6	13
(Illinoian Glacial Till)					0.0098	1.80×10^6	16
					0.0264	1.13×10^6	18
					0.0540	0.79×10^6	19
					0.1083	0.55×10^6	20
					0.2210	0.39×10^6	19
					0.3657	0.30×10^6	18
					0.7353	0.23×10^6	15
P-36	519.7	CL	14.5	122	0.0095	1.97×10^6	12
(Pre-Illinoian Glacial Till)					0.0348	1.40×10^6	12
					0.0667	1.13×10^6	14
					0.1275	0.86×10^6	15
					0.2633	0.63×10^6	16
					0.4468	0.50×10^6	16
					0.9033	0.36×10^6	16
P-38	633.9	ML	8.0	137	0.0055	2.54×10^6	15
(Pre-Illinoian Glacial Till)					0.0148	1.89×10^6	14
					0.0272	1.41×10^6	15
					0.0568	1.00×10^6	18
					0.1069	0.72×10^6	19
					0.1755	0.55×10^6	20
					0.7887	0.23×10^6	
					1.5380	0.17×10^6	18
P-38	588.9	ML	9.2	134	0.0092	0.67×10^6	18
(Illinoian Glacial Till)					0.0160	0.54×10^6	17
					0.0314	0.38×10^6	19
					0.0624	0.27×10^6	18
					0.0936	0.22×10^6	19
					0.1499	0.17×10^6	19
					0.3285	0.11×10^6	20
					0.7944	0.07×10^6	17
					0.6053	0.06×10^6	15

TABLE 2.5-19 (Cont'd)

BORING NUMBER	ELEVATION (ft-in.)	SOIL TYPE	FIELD MOISTURE CONTENT (percent)	DRY DENSITY (lb/ft³)	SINGLE AMPLITUDE SHEAR STRAIN (percent)	MODULUS OF RIGIDITY (lb/ft²)	DAMPING (percent)
P-38	573.9	ML			0.0064	2.69×10^6	8
(Lacustrine Deposit)					0.0141	2.40×10^6	6
					0.0269	2.15×10^6	9
					0.0566	1.66×10^6	10
					0.0839	1.50×10^6	10
					0.1454	1.19×10^6	12
					0.2789	0.90×10^6	16
					0.7643	0.51×10^6	
					1.5905	0.29×10^6	10
P-38	558.9	14.1	128		0.0061	0.99×10^6	17
(Pre-Illinoian Glacial Till)					0.0187	0.72×10^6	17
					0.0305	0.57×10^6	18
					0.0575	0.44×10^6	17
					0.1134	0.32×10^6	18
					0.2225	0.23×10^6	19
					0.4548	0.15×10^6	18
					0.7560	0.12×10^6	18
					1.5318	0.09×10^6	19

TABLE 2.5-20
DYNAMIC TRIAXIAL COMPRESSION TEST DATA
DAM SITE BORINGS
COHESIVE SOILS

BORING NUMBER	ELEVATION (ft-in.)	SOIL TYPE	FIELD MOISTURE CONTENT (percent)	DRY DENSITY (lb/ft³)	SINGLE AMPLITUDE SHEAR STRAIN (percent)	MODULUS OF RIGIDITY (lb/ft²)	DAMPING (percent)
D-6	577.0	ML	8.9	135	0.0128	3.04×10^6	15
					0.0257	2.46×10^6	19
(Illinoian Glacial Till)					0.0489	1.79×10^6	23
					0.1003	1.26×10^6	26
					0.2383	1.02×10^6	24
					0.4767	0.46×10^6	26
					1.2858	0.22×10^6	24
					1.8817	0.20×10^6	24
					3.6380	0.11×10^6	20
					3.5125	0.13×10^6	19
D-11	609.8	ML	6.0	143	0.0262	6.92×10^6	11
					0.0444	5.86×10^6	12
(Illinoian Glacial Till)					0.0523	4.96×10^6	13
					0.131	3.84×10^6	16
					0.263	2.40×10^6	18
					0.339	2.20×10^6	19
					0.664	1.39×10^6	21
					1.043	1.08×10^6	22
					1.340	0.99×10^6	--
D-11	599.8	CL	7.9	139	0.0097	6.01×10^6	34
					0.0237	3.50×10^6	19
(Illinoian Glacial Till)					0.0473	2.47×10^6	21
					0.0947	1.75×10^6	24
					0.2491	0.88×10^6	27
					0.4982	0.54×10^6	27
					0.9964	0.33×10^6	24
					1.8683	0.10×10^6	21
					3.6121	0.12×10^6	16

TABLE 2.5-21
DYNAMIC TRIAXIAL COMPRESSION TEST DATA
DAM SITE BORINGS
COHESIVE SOILS

BORING NUMBER	ELEVATION (ft-in.)	SOIL TYPE	FIELD MOISTURE CONTENT (percent)	DRY DENSITY (lb/ft^3)	RELATIVE DENSITY (percent)	SINGLE\ AMPLITUDE SHEAR STRAIN (percent)	MODULUS OF RIGIDITY (lb/ft^2)	DAMPING (percent)
D-11	464.8	SP	19.5	108	80	0.0043	6.90×10^6	13
						0.0101	5.42×10^6	10
(Mahomet Bedrock Valley Deposit)						0.0222	5.37×10^6	6
						0.0468	4.51×10^6	5
						0.0889	4.20×10^6	5
						0.2402	2.38×10^6	12
						0.4684	1.66×10^6	18
						0.9008	1.31×10^6	20
						1.8016	0.76×10^6	19
						3.3630	0.13×10^6	27
D-11	444.8	SP	19.3	108	80	0.0451	4.01×10^6	11
						0.0938	2.25×10^6	9
(Mahomet Bedrock Valley Deposit)						0.157	1.71×10^6	9
						0.251	1.52×10^6	10
						0.425	1.16×10^6	17
						0.543	1.10×10^6	19
						0.853	0.94×10^6	20
						1.379	0.74×10^6	24
						1.652	0.72×10^6	20
D-11	424.8	SP	19.4	112	82	0.0067	6.63×10^6	12
						0.0087	6.87×10^6	11
(Mahomet Bedrock Valley Deposit)						0.0259	4.80×10^6	8
						0.0506	4.17×10^6	7
						0.0964	3.99×10^6	6
						0.3616	1.58×10^6	11
						0.4821	2.06×10^6	15
						0.9643	1.22×10^6	18
						1.9286	0.87×10^6	19

TABLE 2.5-22
DYNAMIC TRIAXIAL COMPRESSION TEST DATA
ULTIMATE HEAT SINK BORINGS

BORING NUMBER	ELEVATION (ft-in.)	SOIL TYPE	FIELD MOISTURE CONTENT (percent)	DRY DENSITY (lb/ft³)	SINGLE AMPLITUDE SHEAR STRAIN (percent)	MODULUS OF RIGIDITY (lb/ft²)	DAMPING (percent)
H-6	619.3	ML	8.6	136	0.0054	4.03×10^6	8
(Illinoian Glacial Till)					0.0165	2.99×10^6	12
					0.0346	2.26×10^6	16
					0.0757	1.51×10^6	18
					0.1460	1.02×10^6	21
					0.2974	0.66×10^6	22
					0.6083	0.41×10^6	20
					1.0247	0.28×10^6	18
H-14	635.3	ML	9.5	140	0.0134	0.39×10^6	18
(Illinoian Glacial Till)					0.0263	0.28×10^6	18
					0.0407	0.22×10^6	18
					0.0945	0.13×10^6	16
					0.1870	0.08×10^6	16
					0.3019	0.06×10^6	16
					0.6169	0.04×10^6	15
H-20	706.8	ML	8.6	136	0.0076	1.83×10^6	11
(Wisconsinan Glacial Till)					0.0209	1.25×10^6	14
					0.0449	0.85×10^6	15
					0.1015	0.53×10^6	15
					0.1519	0.54×10^6	15
					0.1875	0.44×10^6	15
					0.1875	0.54×10^6	13
					0.3332	0.37×10^6	14
					0.4068	0.31×10^6	14
					0.4068	0.36×10^6	13
					0.7361	0.29×10^6	12

TABLE 2.5-22 (Cont'd)

BORING NUMBER	ELEVATION (ft-in.)	SOIL TYPE	FIELD MOISTURE CONTENT (percent)	DRY DENSITY (lb/ft³)	SINGLE AMPLITUDE SHEAR STRAIN (percent)	MODULUS OF RIGIDITY (lb/ft²)	DAMPING (percent)
					0.8330	0.26×10^6	12
					0.8330	0.29×10^6	12
H-20	686.8	ML			0.0065	1.65×10^6	--
(Interglacial Soil)					0.0157	1.24×10^6	13
					0.0373	0.92×10^6	16
					0.0853	0.62×10^6	20
					0.1630	0.45×10^6	21
					0.3324	0.30×10^6	21
					0.7055	0.18×10^6	21
H-20	672.3	ML	10.5	136	0.0072	2.33×10^6	13
					0.0165	1.84×10^6	14
(Illinoian Glacial Till)					0.O336	1.39×10^6	16
					0.0804	0.82×10^6	19
					0.1608	0.49×10^6	18
					0.3373	0.26×10^6	17
					0.6918	0.13×10^6	15
H-23	677.8	ML	9.1	138	0.0035	5.46×10^6	33
(Illinoian Glacial Till)					0.0096	3.24×10^6	31
					0.0257	1.96×10^6	28
					0.0600	1.20×10^6	26
					0.1281	0.71×10^6	24
					0.2835	0.41×10^6	22
					0.5948	0.22×10^6	--
					0.9967	0.16×10^6	20
H-30	638.5	ML	6.8	145	0.0027	5.85×10^6	8
(Illinoian Glacial Tilll)					0.0084	4.76×10^6	9
					0.0219	3.50×10^6	12

TABLE 2.5-22 (Cont'd)

BORING NUMBER	ELEVATION (ft-in.)	SOIL TYPE	FIELD MOISTURE CONTENT (percent)	DRY DENSITY (lb/ft³)	SINGLE AMPLITUDE SHEAR STRAIN (percent)	MODULUS OF RIGIDITY (lb/ft²)	DAMPING (percent)
					0.0519	2.21×10^6	16
					0.1082	1.34×10^6	--
					0.2597	0.90×10^6	--
					0.5518	0.60×10^6	--
H-36	622.7	ML	9.4	137	0.0051	4.58×10^6	10
(Illinoian Glacial Tilll)					0.0159	3.25×10^6	13
					0.0325	2.55×10^6	16
					0.0716	1.73×10^6	19
					0.1386	1.15×10^6	18
					0.2721	0.76×10^6	18
					0.5881	0.56×10^6	--
					0.5754	0.57×10^6	--
					0.9819	0.50×10^6	--
H-9*	687.1	ML	9.0	126.2	0.0019	5.05×10^6	11
(Interglacial Soil)					0.0246	1.40×10^6	13
					0.1120	0.62×10^6	17
					0.6470	0.19×10^6	20
H-15*	706.3	ML	14.7	118.5	0.0157	0.76×10^6	4
					0.0470	0.48×10^6	17
					0.2310	0.23×10^6	21
					1.1000	0.08×10^6	15
H-32*	656.1	ML	7.9	137.3	0.0076	2.14×10^6	5
					0.0360	0.91×10^6	17
					0.1700	0.51×10^6	25
					0.6920	0.18×10^6	27

* Saturated, consolidated-undrained test

TABLE 2.5-23
DYNAMIC TRIAXIAL COMPRESSION TEST DATA
SECTION E-E' ALONG
NORTH FORK OF SALT CREEK

BORING NUMBER	ELEVATION (ft-in.)	SOIL TYPE	FIELD MOISTURE CONTENT (percent)	DRY DENSITY (lb/ft³)	SINGLE AMPLITUDE SHEAR STRAIN (percent)	MODULUS OF RIGIDITY (lb/ft²)	DAMPING (percent)
D-31	652.7	ML	7.1	140	0.0275	2.61×10^6	9
					0.0546	1.76×10^6	10
(Illinoian Glacial Till)					0.0934	1.49×10^6	9
					0.140	1.31×10^6	10
					0.192	1.10×10^6	13
					0.451	0.79×10^6	19
					0.672	0.59×10^6	19
					1.030	0.42×10^6	21
D-31	643.7	ML	7.6	140	0.0247	1.72×10^6	26
					0.0494	1.13×10^6	29
(Illinoian Glacial Till)					0.0913	0.89×10^6	30
					0.2536	0.42×10^6	29
					0.5072	0.23×10^6	30
					1.3315	0.13×10^6	26
					2.0290	0.12×10^6	20
					3.4239	0.11×10^6	6
					5.0725	0.09×10^6	13
D-31	538.7	ML	9.2	134	0.0266	3.88×10^6	15
					0.0516	2.64×10^6	13
(Illinoian Glacial Till)					0.0840	2.04×10^6	9
					0.1284	2.00×10^6	9
					0.1652	1.21×10^6	15
					0.404	0.89×10^6	19
					0.824	0.53×10^6	21

TABLE 2.5-23 (Cont'd)

BORING NUMBER	ELEVATION (ft-in.)	SOIL TYPE	FIELD MOISTURE CONTENT (percent)	DRY DENSITY (lb/ft³)	SINGLE AMPLITUDE SHEAR STRAIN (percent)	MODULUS OF RIGIDITY (lb/ft²)	DAMPING (percent)
					1.135	0.45×10^6	20
					1.596	0.37×10^6	19
D-31	498.7	CL-ML	17.5	114	0.0121	4.01×10^6	9
					0.0253	2.60×10^6	16
(Mahomet Bedrock Valley Deposit - Silty Alluvium)					0.0456	2.11×10^6	16
					0.1215	1.28×10^6	14
					0.2532	0.84×10^6	16
					0.5063	0.58×10^6	18
					0.9810	0.37×10^6	21
					1.9620	0.19×10^6	25
					3.6076	0.13×10^6	32
					4.8101	0.08×10^6	38

TABLE 2.5-24
DYNAMIC TRIAXIAL COMPRESSION TEST DATA
STRUCTURAL FILL BORROW MATERIAL

BORING NUMBER	ELEVATION (ft)	SOIL TYPE	REMOLDED DRY DENSITY (lb/ft³)	MOISTURE CONTENT (percent)	CONFINING PRESSURE (lb/ft²)	SINGLE AMPLITUDE SHEAR STRAIN (percent)	MODULUS OF RIGIDITY (lb/ft²)	DAMPING (percent)
Combined bulk sample:								
(Salt Creek Alluvium)		SM	123	13.5	6800	0.0074	2.186×10^6	8
			(80% Relative			0.0187	2.200×10^6	9
G-18	663 to 654		Density)			0.0275	2.136×10^6	10
G-19	673 to 663					0.0669	1.555×10^6	12
G-20	657 to 647					0.0983	1.292×10^6	13
						0.1865	0.844×10^6	16
						0.4205	0.356×10^6	17
						0.7758	0.185×10^6	16
		SM	123	13.5	9700	0.0121	3.193×10^6	10
			(80% Relative			0.0159	3.253×10^6	12
			Density)			0.0306	2.609×10^6	12
						0.0606	1.768×10^6	12
						0.2073	0.535×10^6	16
						0.4329	0.233×10^6	17
						0.9041	0.851×10^6	15
		SM	129	12.0	6800	0.0064	3.738×10^6	1
			(90% Relative			0.0174	3.101×10^6	9
			Density)			0.0331	2.496×10^6	12
						0.0726	1.673×10^6	13
						0.2107	0.607×10^6	16
						0.4448	0.244×10^6	16
		SM	129	12.0	9700	0.0074	3.894×10^6	
			(90% Relative			0.0156	3.455×10^6	8
			Density)			0.0280	3.114×10^6	10
						0.0684	2.031×10^6	11
						0.1049	1.648×10^6	12
						0.1827	1.007×10^6	16
						0.4419	0.315×10^6	16
						0.9045	0.130×10^6	12

TABLE 2.5-25
DYNAMIC TRIAXIAL COMPRESSION TEST DATA
REMOLDED WISCONSINAN TILL

BORING NUMBER	ELEVATION (ft)	SOIL TYPE	REMOLDED DRY DENSITY (lb/ft^3)	MOISTURE CONTENT (percent)	CONFINING PRESSURE (lb/ft^2)	SINGLE AMPLITUDE SHEAR STRAIN (percent)	MODULUS OF RIGIDITY (lb/ft^2)	DAMPING (percent)
Combined sample from:								
P-30		CL	120.3	8.7	600	0.0163	1.237×10^6	11
P-32 to						0.0368	1.288×10^6	9
P-35	727 to 692					0.0772	1.108×10^6	10
P-37						0.1152	1.088×10^6	10
P-39						0.2137	0.905×10^6	11
P-42								
		CL	118.6	8.7	1815	0.0063	3.583×10^6	13
						0.0148	3.028×10^6	13
						0.0315	2.363×10^6	15
						0.0755	1.595×10^6	12
						0.1126	1.520×10^6	12
						0.2013	1.248×10^6	12
		CL	121.5	8.5	3630	0.0060	3.893×10^6	11
						0.0128	3.528×10^6	11
						0.0281	2.725×10^6	12
						0.0693	1.975×10^6	14
						0.1124	1.479×10^6	12
						0.2128	1.135×10^6	11
		CL	124.8	8.5	635	0.0078	2.100×10^6	10
						0.0174	1.621×10^6	14
						0.0410	1.032×10^6	12

TABLE 2.5-25 (Cont'd)

BORING NUMBER	ELEVATION (ft)	SOIL TYPE	REMOLDED DRY DENSITY (lb/ft³)	MOISTURE CONTENT (percent)	CONFINING PRESSURE (lb/ft²)	SINGLE AMPLITUDE SHEAR STRAIN (percent)	MODULUS OF RIGIDITY (lb/ft²)	DAMPING (percent)
Combined sample from:						0.0831	9.585×10^6	8
P-30						0.1614	9.794×10^6	9
P-32 to						0.3313	9.273×10^6	9
P-35	727 to 692							
P-37		CL	126.1	8.5	1905	0.0061	4.261×10^6	12
P-39						0.0164	3.273×10^6	11
P-42						0.1581	2.244×10^6	7
						0.3276	2.039×10^6	8
						0.6753	1.641×10^6	10
		CL	126.6	8.5	3810	0.0422	4.107×10^6	--
						0.0809	3.345×10^6	8
						0.1570	2.877×10^6	9
						0.3193	2.332×10^6	10
						0.6738	1.653×10^6	11

TABLE 2.5-26
RESONANT COLUMN TEST DATA
STATION SITE BORING

BORING NUMBER	ELEVATION (ft-in.)	SOIL OR ROCK TYPE	CONFINING PRESSURE (lb/ft²)	MODULUS OF RIGIDITY (lb/ft²)	DRY DENSITY (lb/ft³)	MOISTURE CONTENT (percent)
P-14	609.3	ML	1,000	4.83×10^6	139	7.6
			2,000	6.16×10^6		
(Illinoian Glacial Till)			4,300	7.64×10^6		
			8,600	9.26×10^6		
P-14	468.3	Siltstone	0	67.81×10^6	144	3.0
			5,000	78.76×10^6		
(Pennsylvanian Age-			7,000	78.92×10^6		
Modesto Formation)			10,000	78.92×10^6		
P-18	480.2	Limestone	0	81.22×10^6	167	2.0
			4,000	138.83×10^6		
			6,000	143.49×10^6		
			8,000	146.93×10^6		
P-32	518.4	ML	7,200	5.51×10^6	120	14.4
			14,400	5.99×10^6		
(Pre-Illinoian Lacustrine			21,600	6.05×10^6		
Deposit)			28,800	6.24×10^6		
			36,000	6.24×10^6		
P-36	569.2	ML	7,200	3.33×10^6	133	9.5
(Pre-Illinoian Glacial Till)			14,400	3.80×10^6		
			21,600	3.98×10^6		
			28,800	4.06×10^6		

TABLE 2.5-26 (Cont'd)

BORING NUMBER	ELEVATION (ft-in.)	SOIL OR ROCK TYPE	CONFINING PRESSURE (lb/ft²)	MODULUS OF RIGIDITY (lb/ft²)	DRY DENSITY (lb/ft³)	MOISTURE CONTENT (percent)
P-38	584.9	ML	4,220	8.79×10^6	139	8.2
(Illinoian Glacial Till)			15,850	10.59×10^6		
			24,500	12.06×10^6		
P-38	574.9	ML	7,200	4.48×10^6	132	11.0
(Lacustrine Deposit)			14,400	4.83×10^6		
			21,600	5.09×10^6		
			28,800	5.14×10^6		
P-36	499.5	Limestone	7,200	12.30×10^6	166	-
(Pennsylvanian Age -			14,400	13.26×10^6		
Modeston Formation)			21,600	25.46×10^6		
			28,800	26.58×10^6		
			36,000	27.17×10^6		

TABLE 2.5-27
RESONANT COLUMN TEST DATA
ULTIMATE HEAT SINK BORINGS

BORING NUMBER	ELEVATION (ft-in.)	SOIL OR ROCK TYPE	CONFINING PRESSURE (lb/ft^2)	MODULUS OF RIGIDITY (lb/ft^2)	DRY DENSITY (lb/ft^3)	MOISTURE CONTENT (percent)
H-31	667.1	ML	7,200	7.43×10^6	138*	---*
(Illinoian Glacial Till)			14,400	11.56×10^6	139*	8.7*
H-6	425.8	CH	21,600	3.95×10^6	120	15.2
(Pre-Illinoian			28,800	4.00×10^6	--	--
Lacustrine Deposit)			36,000	4.00×10^6	--	--
H-15	691.3	CL	7,200	2.35×10^6	102*	--
(Interglacial			14,400	3.97×10^6	105*	17.9*
Deposit)						
H-17	641.6	ML	2,045	3.93×10^6	139	8.7
(Illinoian			4,090	4.38×10^6	--	--
Glacial Till)			6,134	4.84×10^6	--	--
			8,180	6.38×10^6	--	--
H-20	716.8	ML	7,200	6.78×10^6	132*	--
(Wisconsinan			14,400	11.39×10^6	134*	10.3*
Glacial Till)						
H-22	540.8	ML	7,200	5.63×10^6	128	12.3
(Pre-Illinoian			14,400	6.65×10^6	--	--
Glacial Till)			21,600	7.12×10^6	--	--
H-30	633.5	ML	6,150	7.07×10^6	141	7.4
(Illinoian			8,194	6.03×10^6	--	--
Glacial Till)			10,238	6.34×10^6	--	--
			12,269	6.91×10^6	--	--
H-36	597.2	ML	8,194	5.67×10^6	139	7.7
(Illinoian			10,238	6.05×10^6	--	--
Glacial Till)			12,283	6.29×10^6	--	--

* Saturated, consolidated-undrained test

TABLE 2.5-28
RESONANT COLUMN TEST DATA*
STRUCTURAL FILL BORROW

BORING NUMBER	ELEVATION (ft)	SOIL OR ROCK TYPE	CONFINING PRESSURE (lb/ft²)	MODULUS OF RIGIDITY (lb/ft²)	DRY DENSITY (lb/ft³)	MOISTURE CONTENT (percent)
Combined bulk		SM	4800	29.26×10^5	123.1	Dry
sample from:					(78%	
			6200	41.63×10^5	Relative	
G-18	663-654				Density)	
G-19	673-663		7400	47.54×10^5		
G-20	657-647		9000	53.10×10^5		
		SM	4800	35.10×10^5	128.9	Dry
(Salt Creek					(90%	
Alluvium)			6200	41.13×10^5	Relative	
					Density)	
			7400	47.96×10^5		
			9000	54.65×10^5		
			9648	57.03×10^5		

* Tests performed at 0.001 percent shear strain.

TABLE 2.5-29
RESONANT COLUMN TEST DATA
REMOLDED WISCONSINAN TILL*

BORING NUMBER	ELEVATION	SOIL OR ROCK TYPE	CONFINING PRESSURE (lb/ft²)	MODULUS OF RIGIDITY (lb/ft²)	DRY DENSITY (lb/ft³)	COMPACTION EFFORT** (percent)	MOISTURE CONTENT (percent)
Combined sample from:							
P-30		CL	600	3.56×10^6	119.9	89.2	8.7
P-32 to			1814	5.26×10^6			
P-35	727 to 692		3628	6.34×10^6			
P-37			5440	7.13×10^6			
P-39							
P-42							
		CL	1814	5.48×10^6	124.3	92.5	8.5
			3628	7.08×10^6			
			5440	7.96×10^6			

* As compacted.
** AASHO Test Designation T-180.

TABLE 2.5-30
SHOCKSCOPE TEST DATA
STATION SITE BORINGS

BORING NUMBER	ELEVATION (ft-in)	SOIL OR ROCK TYPE	GEOLOGIC UNIT	CONFINING PRESSURE (lb/ft²)	VELOCITY OF COMPRESSIONAL WAVE PROPAGATION (ft/sec)
P-14	599.3	ML	Illinoian Glacial Till	0	6666
P-14	529.3	CL	Pre-Illinoian Glacial Till	0	6553
P-14	460.3	Shale	Pennsylvanian Age-Modesto Formation	0	6423
P-18	704.2	ML	Wisconsinan Glacial Till	0	6321
P-18	485.2	Limestone	Pennsylvanian Age-Modesto Formation	0	7776
P-18	468.2	Shale	Pennsylvanian Age-Modesto Formation	0	5280

TABLE 2.5-31
LABORATORY PERMEABILITY TEST DATA
STATION SITE BORINGS

BORING	ELEVATION (ft-in.)	SOIL TYPE	GEOLOGIC UNIT	TYPE OF TEST	FIELD MOISTURE CONTENT (percent)	FIELD DRY DENSITY (lb/ft^3)	AVERAGE COEFFICIENT OF PERMEABILITY AT 20° C K (cm/sec)
P-14	654.8	ML	Illinoian Glacial Till	Falling Head	9.5	129	2.5×10^{-8}
P-14	579.8	ML	Illinoian Glacial Till	Falling Head	8.1	139	9.5×10^{-9}
P-18	683.7	ML.SM	Illinoian Glacial Till	Falling Head	10.3	131	2.3×10^{-7}

TABLE 2.5-32
LABORATORY PERMEABILITY TEST DATA
DAM SITE BORINGS

BORING	ELEVATION (ft-in.)	SOIL TYPE	GEOLOGIC UNIT	TYPE OF TEST	FIELD MOISTURE CONTENT (percent)	FIELD DRY DENSITY (lb/ft^3)	AVERAGE COEFFICIENT OF PERMEABILITY AT 20° C K (cm/sec)	ESTIMATED POROSITY (percent)
D-3	626.2	ML	Illinoian Glacial Till	Falling Head	7.5	144	3.9×10^{-9}	16.8
D-10	627.0	ML	Illinoian Glacial Till	Falling Head	7.2	131	1.0×10^{-8}	16.3
D-13	676.4	SP	Interglacial Zone	Constant Head	24.8	94	1.8×10^{-4}	40.0
D-13	661.4	SP SW	Interglacial Zone	Constant Head	6.4	105	4.7×10^{-3}	14.8
D-13	632.0	ML	Illinoian Glacial Till	Falling Head	7.3	142	3.8×10^{-9}	16.4
D-24	631.0	ML	Salt Creek Alluvium	Falling Head	7.4	123	1.8×10^{-8}	16.5
D-34	664.8	SP GP	Interglacial Zone	Constant Head	6.2	112	2.3×10^{-3}	14.3
D-34	649.8	SP GP	Interglacial Zone	Constant Head	17.5	118	2.0×10^{-4}	32.0

TABLE 2.5-32 (Cont'd)

BORING	ELEVATION (ft-in.)	SOIL TYPE	GEOLOGIC UNIT	TYPE OF TEST	FIELD MOISTURE CONTENT (percent)	FIELD DRY DENSITY (lb/ft³)	AVERAGE COEFFICIENT OF PERMEABILITY AT 20° C K (cm/sec)	ESTIMATED POROSITY (percent)
D-34	629.8	ML	Illinoian Glacial Till	Falling Head	7.8	138	6.5×10^{-9}	17.4
D-37	663.7	SP SW	Interglacial Zone	Constant Head	12.2	116	3.0×10^{-3}	24.7
D-37	643.7	ML CL	Illinoian Glacial Till	Falling Head	11.7	134	1.3×10^{-8}	24.0

TABLE 2.5-33
LABORATORY PERMEABILITY TEST DATA
REMOLDED SAMPLES***

BORING	ELEVATION (ft-in.)	SOIL TYPE*	GEOLOGIC UNIT	TYPE OF TEST	REMOLDED DATA			AVERAGE COEFFICIENT OF PERMEABILITY AT 20° C K (cm/sec)
					MOISTURE CONTENT (percent)	DRY DENSITY (lb/ft³)	COMPACTION EFFORT** (percent)	
S-10	702.6 to 697.6	CL	Wisconsinan Glacial Till	Falling Head	13.6	126	97.3	8.2×10^{-9}
S-10	702.6 to 697.6	CL	Wisconsinan Glacial Till	Falling Head	12.4	125	96.5	2.0×10^{-8}
S-14	727.2 to 720.2	CL	Wisconsinan Glacial Till	Falling Head	16.8	109	91.5	1.6×10^{-8}
S-14	727.2 to 720.2	CL	Wisconsinan Glacial Till	Falling Head	11.0	125	105.0	1.0×10^{-8}

* See Figure 2.5-355 for definition of soil type symbols.

** AASHO Test Designation T-180.

*** As compacted.

TABLE 2.5-34
RELATIVE DENSITY TEST DATA

BORING NUMBER	ELEVATION (ft-in.)	GEOLOGIC UNIT	MINIMUM DRY DENSITY (lb/ft³)	MAXIMUM DRY DENSITY (lb/ft³)	IN SITU RELATIVE DENSITY (percent)
D-11	473.8	Mahomet Bedrock Valley Deposit	92	113 (Wet Method)	--
D-11	424.8	Mahomet Bedrock Valley Deposit	91	118 (Wet Method)	82

TABLE 2.5-35
RESULTS OF STRESS-CONTROLLED CYCLIC TRIAXIAL (LIQUEFACTION) TESTS

SAMPLE NUMBER	SOIL DESCRIPTION*	DRY DENSITY	MOLDING MOISTURE CONTENT (percent)	PRINCIPAL CONSOLIDATION STRESS RATIO, Kc	EFFECTIVE CONFINING PRESSURE (psf)	CYCLIC STRESS RATIO $\Delta\sigma_v / 2\bar{\sigma}_c$	SKEMPTON'S PORE PRESSURE PARAMETER B	NUMBER OF CYCLES REQUIRED TO CAUSE: INITIAL LIQUE-FACTION***	NUMBER OF CYCLES REQUIRED TO CAUSE: 5% STRAIN**
Type B Granular Structural Fill Material									
S-1	Light brown fine	126.5	7.0	1.0	6800	0.403	0.95	33	42
S-2	to coarse sand	126.4	7.0	1.0	6800	0.504	0.95	20	33
S-3	with gravel	127.2	7.0	1.0	6800	0.655	0.96	13	12
S-4		127.4	7.0	1.0	6800	0.302	0.95	50	55

* For soil properties see Figure 2.5-348.

** Double Amplitude Axial Strain.

*** Initial liquefaction is defined as when the increase in pore pressure is equal to the effective confining pressure.

TABLE 2.5-36
AVERAGE SOIL PROPERTIES OF BORROW MATERIALS
FOR THE ULTIMATE HEAT SINK

SOIL PROPERTIES	
Soil Type	Brown and gray clayey silt to silty clay with trace to some sand and trace fine gravel.
	AVERAGE
Liquid Limit (%)	21.7
Plasticity Index (%)	7.7
Compaction Test (ASTM D1557)	
Maximum Dry Density (lb/ft^3)	133.8
Optimum Moisture Content (%)	8.4

TABLE 2.5-37
AVERAGE SOIL PROPERTIES OF COHESIVE BORROW MATERIALS
FOR THE MAIN PLANT AND SCREEN HOUSE

SOIL PROPERTIES	
Soil Type	Brown and gray silty clay with trace to some sand and trace fine gravel.
	AVERAGE
Liquid Limit (%)	23.4
Plasticity Index (%)	8.9
Compaction Test (ASTM D1557)	
Maximum Dry Density (lb/ft^3)	133.1
Optimum Moisture Content (%)	8.7

TABLE 2.5-38
FIELD PERMEABILITY TESTS

BORING	GROUND SURFACE ELEVATION (ft-in.)	ZONE OF PERCOLATION ELEVATION (ft-in.)	GEOLOGIC UNIT	AVERAGE COEFFICIENT OF PERMEABILITY, k (cm/sec)	ESTIMATED POROSITY (percent)
D-19	658.9	625.0 to 620.9	Illinoian Till	1.4×10^{-5}	26.7
D-23	655.8	630.8 to 624.3	Illinoian Till	6.1×10^{-6}	24.5
E-1B	733.0	703.0 to 693.0	Wisconsinan Till	1.5×10^{-6}	--
P-37	741.5	726.1 to 701.1	Wisconsinan Till	2.6×10^{-6}	25.7

TABLE 2.5-39
OBSERVED SURFACE WAVE CHARACTERISTICS
STATION SITE

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	Transverse - Radial	11 - 12	240	2900	6
2	Sezawa M_2	Transverse - Vertical	6 - 7	240	1500 - 1600	5
3	Sezawa M_2	Transverse - Radial	7	120 - 140	900	8 - 10
4	(*Rayleigh?)	Vertical - Radial	7	80 - 100	greater than 600 - 700	--

* There is an indication of a Rayleigh wave having a velocity greater than 600 - 700 feet per second and a frequency of 7 Hz. The initial motion of this wave could not be determined from the field data.

TABLE 2.5-40
OBSERVED SURFACE WAVE CHARACTERISTICS
DAM SITE

OBSERVED WAVE	WAVE TYPE	PREDOMINANT PARTICLE MOTION	PREDOMINANT FREQUENCY (Hz)	APPARENT WAVE LENGTH (ft)	APPARENT VELOCITY (ft/sec)	OBSERVED LENGTH OF WAVE TRAIN (cycles)
1	Sezawa M_2	Radial - Vertical	10	190	1900	7
2	Rayleigh	Radial - Transverse	8	125	1000	6
3	?*	Radial - Transverse	7 - 8	50 - 60	400 - 500	8

* This wave displays little motion on the vertical component; however, this could be the result of normal attenuation, and not the indication of a Love wave.

TABLE 2.5-41
OBSERVED SURFACE WAVE CHARACTERISTICS
SECTION E - E' ALONG
NORTH FORK OF SALT CREEK

OBSERVED WAVE	WAVE TYPE	PREDOMINANT PARTICLE MOTION	PREDOMINANT FREQUENCY (Hz)	APPARENT WAVE LENGTH (ft)	APPARENT VELOCITY (ft/sec)	OBSERVED LENGTH OF WAVE TRAIN (cycles)
1	Sezawa M_2	Verticle - Transverse	10	150	1500	8 - 10
2	Sezawa M_2	Radial - Transverse	10	70	700	6

TABLE 2.5-42
AMBIENT GROUND MOTION MEASUREMENTS

(June 6 and 13, 1972)

				GROUND MOTION**		
LOCATION	TIME	FREQUENCY (Hz)	GAIN* (x 100)	RAD	VERT	TRANS***
Station site:			V x 5	0.06	0.06	0.06
Boring	9:30 AM	10-12, 50,150	A x 20	0.12	0.08	0.10
P-18	June 6		D x 20	0.0015	0.002	0.002
Station site:			V x 20	0.05	0.04	0.08
Boring	5:00 PM	10-12, 50	A x 20	0.058	0.058	0.067
P-28	June 6		D x 20	0.0015	0.0005	0.0010
Dam site:						
Boring	10:30 AM	8-10,33-1/3,	V x 20	0.07	0.10	0.11
D-11	June 6	50, 100	A x 20	0.067	0.083	0.12
			D x 20	0.003	0.0005	0.001
Section E - E' along the North Fork of Salt Creek:						
Boring	2:00 PM	4, 25,33-1/3,	V x 20	0.08	0.08	0.09
D-31	June 13	150	A x 20	0.017	0.05	0.033
			D x 20	0.001	0.001	0.001

* V = velocity (in./sec)
A = acceleration (in./sec/sec)
D = displacement (in.).

** All values are x 10^{-3}.

*** RAD = radial
VERT = vertical
TRANS = transverse.

TABLE 2.5-43
LABORATORY COMPRESSIONAL WAVE VELOCITY TABULATION

BORING NUMBER	ELEVATION (depth)	SOIL OR ROCK TYPE	GEOLOGIC UNIT	DATA SOURCE	CONFINING PRESSURE (lb/ft²)	SHEAR STRAIN %	VELOCITY (ft/sec)
P-14	713.8 (24.5)	ML	Wisconsinan Glacial Till	Dynamic Triaxial Compression Test	2,500	0.014	1320
						0.0264	1220
						0.0521	1090
						0.1268	880
						0.2508	740
						0.5191	690
						1.2540	390
						1.9153	358
						2.5205	337
P-15	691.8 (44.5)	ML	Interglacial Zone	Dynamic Triaxial Compression Test	3,000	0.0144	1280
						0.0277	1165
						0.0531	1085
						0.1037	920
						0.2529	740
						0.5059	605
						1.2647	456
						2.5170	354
						3.6941	290
						5.0841	260
P-14	624.3 (114.0)	CL	Illinoian Glacial Till	Dynamic Triaxial Compression Test	9,000	0.0127	1830
						0.0262	1610
						0.0508	1375
						0.1272	1085
						0.2502	930
						0.5077	775
						0.9921	675
						1.9841	550
						3.8697	483

TABLE 2.5-43 (Cont'd)

BORING NUMBER	ELEVATION (depth)	SOIL OR ROCK TYPE	GEOLOGIC UNIT	DATA SOURCE	CONFINING PRESSURE (lb/ft²)	SHEAR STRAIN %	VELOCITY (ft/sec)
P-22	659.5 (74.5)	ML	Illinoian Glacial Till	Dynamic Triaxial Compression Test	6,000	0.0125	1840
						0.0251	1560
						0.0494	1380
						0.1245	1130
						0.2462	950
						0.4803	700
						0.7217	676
						0.9721	620
D-6	577.00 (79.0)	ML	Illinoian Glacial Till	Dynamic Triaxial Compression Test	6,000	0.0128	1975
						0.0257	1790
						0.0489	1530
						0.1003	1275
						0.2383	1115
						0.4767	770
						1.2858	535
						1.8817	510
						3.5125	412
						3.3680	378
D-11	599.8 (54.0)	CL	Illinoian Glacial Till	Dynamic Triaxial Compression Test	4,000	0.0097	2790
						0.0237	2140
						0.0473	1790
						0.0947	1510
						0.2491	1070
						0.4982	840
						0.9964	650
						1.8683	360
						3.6121	395

TABLE 2.5-43 (Cont'd)

BORING NUMBER	ELEVATION (depth)	SOIL OR ROCK TYPE	GEOLOGIC UNIT	DATA SOURCE	CONFINING PRESSURE (lb/ft²)	SHEAR STRAIN %	VELOCITY (ft/sec)
D-11	609.8	ML	Illinoian	Dynamic	4,000	0.0262	3000
	(44.0)		Glacial	Triaxial		0.0444	2760
			Till	Compression		0.0523	2540
				Test		0.131	2230
						0.263	1760
						0.339	1690
						0.664	1340
						1.043	1180
						1.340	1130
D-11	464.8	SP	Mahomet	Dynamic	9,000	0.0043	3200
	(189.0)		Bedrock	Triaxial		0.0101	2840
			Valley	Compression		0.0222	2820
			Deposit	Test		0.0468	2590
						0.0889	2500
						0.2402	1880
						0.4684	1530
						0.9008	1370
						1.8016	1060
						3.363	440
D-11	444.8	SP	Mahomet	Dynamic	9,000	0.0451	2440
	(209.0)		Bedrock	Triaxial		0.0938	1830
			Valley	Compression		0.159	1590
			Deposit	Test		0.251	1510
						0.425	1310
						0.543	1275
						0.853	1184
						1.379	1050
						1.652	1030

TABLE 2.5-43 (Cont'd)

BORING NUMBER	ELEVATION (depth)	SOIL OR ROCK TYPE	GEOLOGIC UNIT	DATA SOURCE	CONFINING PRESSURE (lb/ft²)	SHEAR STRAIN %	VELOCITY (ft/sec)
D-31	498.7	CL	Mahomet	Dynamic	10,000	0.0121	2370
	(169.0)		Bedrock	Triaxial		0.0253	1920
			Valley	Compression		0.0456	1730
			Deposit	Test		0.1215	1350
						0.2532	1090
						0.5063	900
						0.9810	720
						1.962	520
						3.6076	425
						4.8101	316
D-11	424.8	SP	Mahomet	Dynamic	11,000	0.0067	2930
	(229.0)		Bedrock	Triaxial		0.0087	2990
			Valley	Compression		0.0259	2500
			Deposit	Test		0.0506	2330
						0.0964	2280
						0.3616	1450
						0.4821	1630
						0.9643	1260
						1.9286	1060
D-31	538.7	ML	Illinoian	Dynamic	10,000	0.0266	2240
	(129.0)		Glacial	Triaxial		0.0516	1850
			Till	Compression		0.0840	1640
				Test		0.1284	1610
						0.1652	1255
						0.404	1072
						0.824	825
						1.135	761
						1.596	690

TABLE 2.5-43 (Cont'd)

BORING NUMBER	ELEVATION (depth)	SOIL OR ROCK TYPE	GEOLOGIC UNIT	DATA SOURCE	CONFINING PRESSURE (lb/ft²)	SHEAR STRAIN %	VELOCITY (ft/sec)
D-31	643.7 (24.0)	ML	Illinoian Glacial Till	Dynamic Triaxial Compression Test	2,500	0.0247	1480
						0.0494	1190
						0.0913	1060
						0.2536	727
						0.5072	537
						1.3315	407
						2.0290	398
						3.4239	360
						5.0725	338
D-31	652.7 (15.0)	ML	Illinoian Glacial Till	Dynamic Triaxial Compression Test	2,500	0.0275	1840
						0.0546	1520
						0.0934	1390
						0.140	1300
						0.192	1200
						0.451	1020
						0.672	870
						1.030	736
P-14	599.3 (139.0)	ML	Illinoian Glacial Till	Shockscope Test	0		6666
P-14	529.3 (209.0)	CL	Pre-Illinoian Glacial Till	Shockscope Test	0		6553
P-14	460.3 (278.0)	Shale		Shockscope Test	0		6423

TABLE 2.5-43 (Cont'd)

BORING NUMBER	ELEVATION (depth)	SOIL OR ROCK TYPE	GEOLOGIC UNIT	DATA SOURCE	CONFINING PRESSURE (lb/ft²)	SHEAR STRAIN %	VELOCITY (ft/sec)
P-18	704.2 (34.0)	ML	Wisconsinan Glacial Till	Shockscope Test	0		6321
P-18	485.2 (253.0)	Lime-stone		Shockscope Test	0		7776
P-18	468.2 (270.0)	Shale		Shockscope Test	0		5280

TABLE 2.5-44
LABORATORY SHEAR WAVE VELOCITY TABULATION

BORING NUMBER	ELEVATION (depth)	SOIL OR ROCK TYPE	GEOLOGIC UNIT	DATA SOURCE	CONFINING PRESSURE (lb/ft²)	SHEAR STRAIN %	VELOCITY (ft/sec)
P-14	468.3	Sandstone		Resonant	10,000	0.077	4140
	(270.0)			Column	7,000	0.077	4140
					5,000	0.077	4150
					1.4	0.089	3840
P-18	480.2	Sandstone		Resonant	8,000	0.044	5270
	(258.0)			Column	6,000	0.047	5210
					4,000	0.046	5130
					1.4	0.079	3920
P-14	609.3	ML	Illinoian	Resonant	8,640	0.672	1410
	(129)		Glacial	Column	4,320	0.0857	1280
			Till		2,016	0.1048	1150
					1,008	0.1234	1040

TABLE 2.5-45
FIELD COMPRESSIONAL WAVE VELOCITY TABULATION

VELOCITY (ft/sec)	SOURCE	MATERIAL TYPE	DEPTH (feet)
2,000	Seismic Line 1	Low-velocity surface layer	0-14
5,750	Seismic Line 1	Wisconinan Till	14-56
7,500	Seismic Line 1	Illinoian Glacial Till	56-233
10,000	Seismic Line 1	Top of bedrock	233+
2,000	Seismic Line 2	Low-velocity surface layer	0-15
5,700	Seismic Line 2	Wisconinan Till	15-46
7,500	Seismic Line 2	Illinoian Glacial Till	46-238
10,000	Seismic Line 2	Top of bedrock	238+
2,000	Seismic Line 3	Low-velocity surface layer	0-15
5,700	Seismic Line 3	Wisconinan Till	15-55
7,500	Seismic Line 3	Illinoian Glacial Till	55-243
9,750	Seismic Line 3	Top of bedrock	243+
2,000	Seismic Line 4	Low-velocity surface layer	0-16
5,500	Seismic Line 4	Wisconinan Till	16-55
7,250	Seismic Line 4	Illinoian Glacial Till	55-243
10,250	Seismic Line 4	Top of bedrock	243+
2,000	Seismic Line 5	Low-velocity surface layer	0-12
5,750	Seismic Line 5	Wisconinan Till	12-48
7,500	Seismic Line 5	Illinoian Glacial Till	48-240
10,500	Seismic Line 5	Top of bedrock	240+
2,000	Seismic Line 6	Low-velocity surface layer	0-18
7,300	Seismic Line 6	Wisconinan Till	18-150
5,800	Seismic Line 6	Illinoian Glacial Till	150-290
10,600	Seismic Line 6	Top of bedrock	290+

TABLE 2.5-45 (Cont'd)

VELOCITY (ft/sec)	SOURCE	MATERIAL TYPE	DEPTH (feet)
2,000	Seismic Line 7	Low-velocity surface layer	0-14
7,500	Seismic Line 7	Illinoian Glacial Till	14-195
6,000	Seismic Line 7	Bedrock Valley Outwash Deposit	195-305
9,800	Seismic Line 7	Top of bedrock	305+
2,875	Uphole P-14	Low-velocity surface layer	0-10
4,800	Uphole P-14	Wisconinan Till	10-57
7,400	Uphole P-14	Illinoian Glacial Till	57-237
12,000	Uphole P-14	Top of bedrock	237+
6,800	Uphole D-11	Illinoian Glacial Till	0-275
7,000	Uphole D-31	Illinoian Glacial Till	0-275

TABLE 2.5-46
FIELD SHEAR WAVE VELOCITY TABULATION

ESTIMATED VELOCITY (ft/sec)	SOURCE	MATERIAL TYPE	DEPTH (feet)
900*	Geophysical P-14	Low-velocity surface layer	0-16
1100	Geophysical P-14	Wisconsinan till	16-47
2100	Geophysical P-14	Illinoian glacial till	47-237
5700	Geophysical P-14	Top of bedrock	237+
900*	Geophysical D-11	Salt Creek alluvium	0-18
2000-2100	Geophysical D-11	Illinoian glacial till	18-150
1800	Geophysical D-11	Bedrock valley outwash deposit	150-290
5700	Geophysical D-11	Top of bedrock	290+
900*	Geophysical D-31	Salt Creek alluvium	0-14
2100	Geophysical D-31	Illinoian glacial till	14-195
1800	Geophysical D-31	Bedrock valley	195-305
5300-5500	Geophysical D-31	Top of bedrock	305+

* Measured.

TABLE 2.5-47
DETAILS OF STRUCTURE FOUNDATIONS AND BEARING STRATA

	FOUNDATION MAT		BEARING STRATUM		
STRUCTURE	APPROXIMATE SIZE, L x B (ft x ft)	APPROXIMATE BOTTOM ELEVATION (ft)	TYPE*	COMPACTED THICKNESS (ft)	SOIL UNDERLYING BEARING STRATUM
Containment	130 dia.	702.0	B	22.0	A
Fuel Building	182 x 151	702.3	B	22.0	A
Auxiliary Bldg.	178 x 122	697.8	B	17.5	A
Radwaste, machine Shop, and Off-Gas Bldg.	232 x 321	693.0	B	12.0	A
Service Bldg.	96 x 195	732.0	B	30.0	A
Diesel Generator and HVAC	222 x 106	703.0	B	22.0	A
Control	219 x 100	693.0	B	12.0	A
Turbine and Heater Bay	185 x 315	702.0	B	22.0	A
Circulating Water Screen House	238 x 176	653.0	A	-	-
Ultimate Heat Sink Outlet Structure	43 x 34	669.0	C	7.0	B,A

* A: Existing Illinoian glacial till.

B: Type B structural fill (onsite granular-type material). Minimum placement relative density = 85% as determined by ASTM D2049-69 test method.

C: Fly ash mixture.

TABLE 2.5-48

PARAMETERS FOR ANALYSIS OF ROCK-SOIL-STRUCTURE INTERACTION

		COHESIONLESS SOIL	COHESIVE SOILS				
		COMPACTED STRUCTURAL FILL	RECOMPACTED WISCONSINAN GLACIAL TILL OF WEDRON FORMATION TYPE A MATERIAL (AS COMPACTED)	RECOMPACTED WISCONSINAN GLACIAL TILL OF WEDRON FORMATION TYPE A MATERIAL (SATURATED)	LOESS	WISCONSINAN GLACIAL TILL OF WEDRON FORMATION	INTERGLACIAL DEPOSITS
DENSITY (pcf):	Dry density	123	127	128	101	118	115
	Wet density	132	141	144	120	137	131
POISSON'S RATIO:	Dynamic	0.40	0.40	0.40	0.37	0.48	0.48
	Static	0.30	0.40	0.40	0.40	0.40	0.40
STATIC MODULUS OF ELASTICITY (Es)							
In-situ modulus (psf)		--	8.0×10^5	2.0×10^5	2.0×10^5	13.1×10^5	15.1×10^5
Increase with surcharge $dEs/d\sigma'_m$ (psf/psf)		350	0	0	0	0	0
DYNAMIC MODULUS OF ELASTICITY (psf)							
Single amplitude Shear strain	= 1.0%	$22{,}000\ (\sigma'_m)^{½}$	11×10^5	3×10^5	3×10^5	12×10^5	9×10^5
	= 0.1%	$90{,}000\ (\sigma'_m)^{½}$	39×10^5	8×10^5	8×10^5	36×10^5	33×10^5
	= 0.01%	$207{,}000\ (\sigma'_m)^{½}$	98×10^5	34×10^5	33×10^5	80×10^5	80×10^5
	= 0.001%	$271{,}000\ (\sigma'_m)^{½}$	148×10^5	76×10^5	74×10^5	130×10^5	130×10^5
	= 0.0001%	$280{,}000\ (\sigma'_m)^{½}$	162×10^5	95×10^5	93×10^5	160×10^5	160×10^5
STATIC MODULUS OF RIGIDITY (Gs)							
In-situ modulus (psf)		--	3.0×10^5	0.7×10^5	0.7×10^5	4.7×10^5	5.4×10^5
Increase with surcharge $dGs/d\sigma'_m$ (psf/psf)		135	0	0	0	0	0

TABLE 2.5-48 (Continued)

	COHESIONLESS SOIL	COHESIVE SOILS				
	COMPACTED STRUCTURAL FILL	RECOMPACTED WISCONSINAN GLACIAL TILL OF WEDRON FORMATION TYPE A MATERIAL (AS COMPACTED)	RECOMPACTED WISCONSINAN GLACIAL TILL OF WEDRON FORMATION TYPE A MATERIAL (SATURATED)	LOESS	WISCONSINAN GLACIAL TILL OF WEDRON FORMATION	INTERGLACIAL DEPOSITS
DYNAMIC MODULUS OF RIGIDITY (psf)						
Single amplitude						
Shear strain = 1.0%	$8,000 (\sigma'_m)^{1/2}$	4×10^5	1×10^5	1×10^5	4×10^5	3×10^5
= 0.1%	$32,000 (\sigma'_m)^{1/2}$	14×10^5	3×10^5	3×10^5	12×10^5	11×10^5
= 0.01%	$74,000 (\sigma'_m)^{1/2}$	35×10^5	12×10^5	12×10^5	27×10^5	27×10^5
= 0.001%	$97,000 (\sigma'_m)^{1/2}$	53×10^5	27×10^5	27×10^5	44×10^5	44×10^5
= 0.0001%	$100,000 (\sigma'_m)^{1/2}$	58×10^5	34×10^5	34×10^5	54×10^5	54×10^5
DAMPING						
Percent of critical damping						
Single amplitude						
Shear strain = 1.0%	16	20	20	20	20	20
= 0.1%	14	9	15	15	9	9
= 0.01%	6	5	10	10	5	5
= 0.001%	2	3	6	6	3	3
= 0.0001%	1	2.5	4	4	2.5	2.5

TABLE 2.5-48 (Continued)

	COHESIONLESS SOIL		COHESIVE SOILS			COHESIONLESS SOIL	
	SALT CREEK ALLUVIUM	INTERGLACIAL SAND DEPOSITS	ILLINOIAN GLACIAL TILL	LACUSTRINE DEPOSITS	PRE-ILLINOIAN DEPOSITS	PRE-ILLINOIAN DEPOSITS	ROCK*
DENSITY (pcf): Dry density	100	108	138	123	130	107	156
Wet density	125	120	150	134	145	126	159
POISSON'S RATIO: Dynamic	0.37	0.40	0.46	0.47	0.47	0.40	0.29
Static	0.40	0.40	0.35	0.35	0.35	0.40	0.29
STATIC MODULUS OF ELASTICITY (Es)							
In-situ modulus (psf)	--	--	43.6×10^5	24.9×10^5	42.4×10^5	110×10^5	0.7 to 3.8×10^8
Increase with surcharge $dEs/d\sigma'_m$ (psf/psf)	150	260	0	0	0	1100	0
DYNAMIC MODULUS OF ELASTICITY (psf)							
Single amplitude Shear strain = 1.0%	$2{,}700 (\sigma'_m)^{1/2}$	$4{,}200 (\sigma'_m)^{1/2}$	23×10^5	24×10^5	24×10^5	$28{,}200 (\sigma'_m)^{1/2}$	3.6 to 7.8×10^8
= 0.1%	$11{,}000 (\sigma'_m)^{1/2}$	$17{,}000 (\sigma'_m)^{1/2}$	88×10^5	76×10^5	76×10^5	$95{,}000 (\sigma'_m)^{1/2}$	0
= 0.01%	$44{,}000 (\sigma'_m)^{1/2}$	$62{,}000 (\sigma'_m)^{1/2}$	292×10^5	226×10^5	226×10^5	$174{,}000 (\sigma'_m)^{1/2}$	
= 0.001%	$52{,}000 (\sigma'_m)^{1/2}$	$81{,}000 (\sigma'_m)^{1/2}$	496×10^5	338×10^5	338×10^5	$218{,}000 (\sigma'_m)^{1/2}$	
= 0.0001%	$280{,}000 (\sigma'_m)^{1/2}$	$84{,}000 (\sigma'_m)^{1/2}$	584×10^5	412×10^5	412×10^5	$238{,}000 (\sigma'_m)^{1/2}$	
STATIC MODULUS OF RIGIDITY (Gs)							
In-situ modulus (psf)	--	--	16.1×10^5	9.2×10^5	15.7×10^5	40×10^5	0.3 to 1.5×10^8
Increase with surcharge $dGs/d\sigma'_m$ (psf/psf)	54	93	0	0	0	392	0

TABLE 2.5-48 (Continued)

	COHESIONLESS SOIL		COHESIVE SOILS			COHESIONLESS SOIL	
	SALT CREEK ALLUVIUM	INTERGLACIAL SAND DEPOSITS	ILLINOIAN GLACIAL TILL	LACUSTRINE DEPOSITS	PRE-ILLINOIAN DEPOSITS	PRE-ILLINOIAN DEPOSITS	ROCK*
DYNAMIC MODULUS OF RIGIDITY (psf)							
Single amplitude							
Shear strain = 1.0%	$1{,}000\,(\sigma'_m)^{1/2}$	$1{,}500\,(\sigma'_m)^{1/2}$	8×10^5	8×10^5	4×10^5	$10{,}500\,(\sigma'_m)^{1/2}$	1.4 to 3.0×10^8
= 0.1%	$4{,}000\,(\sigma'_m)^{1/2}$	$6{,}000\,(\sigma'_m)^{1/2}$	30×10^5	26×10^5	26×10^5	$34{,}000\,(\sigma'_m)^{1/2}$	0
= 0.01%	$16{,}000\,(\sigma'_m)^{1/2}$	$22{,}000\,(\sigma'_m)^{1/2}$	100×10^5	77×10^5	77×10^5	$62{,}000\,(\sigma'_m)^{1/2}$	
= 0.001%	$19{,}000\,(\sigma'_m)^{1/2}$	$29{,}000\,(\sigma'_m)^{1/2}$	170×10^5	115×10^5	115×10^5	$78{,}000\,(\sigma'_m)^{1/2}$	
= 0.0001%	$20{,}000\,(\sigma'_m)^{1/2}$	$30{,}000\,(\sigma'_m)^{1/2}$	200×10^5	140×10^5	140×10^5	$85{,}000\,(\sigma'_m)^{1/2}$	
DAMPING							
Percent of critical damping							
Single amplitude							
Shear strain = 1.0%	21	28	22	20	20	20	1 to 2
= 0.1%	10	13	16	9	12	10	
= 0.01%	3	4	7.5	4.5	7.5	3	
= 0.001%	1	1.5	4.	3	4.	2	
= 0.0001%	0.5	0.5	3	2.5	3	1	

* These values are valid for strain levels on the order of 10^{-4} to 10^{-5} percent.

Notes:
1. σ'_m = mean effective stress (psf).
2. The static modulus of elasticity values for cohesive soils were calculated based on the constrained modulus derived from the reloading portion of the consolidation curve.
3. Pre-Illinoian cohesive deposits include glacial and lacustrine deposits.
4. Pre-Illinoian cohesionless deposits include Mahomet Valley deposits.
5. The selected parameters reflect both the results of geophysical and laboratory tests performed during this investigation and results published and previously developed for similar soils.

TABLE 2.5-49
SUMMARY OF LIQUEFACTION ANALYSES

ELEVATION* (FEET)	EFFECTIVE OVERBURDEN PRESSURE $\sigma o \left(kips/ft^2\right)$	AVERAGE CYCLIC SHEAR STRESS FOR 10 CYCLES $\tau liq \left(kips/ft^2\right)$	AVERAGE CYCLIC SHEAR STRESS CAUSING LIQUE-FACTION IN 10 CYCLES $\tau liq \left(kips/ft^2\right)$	FACTOR OF SAFETY WITH RESPECT TO INITIAL LIQUEFACTION $\tau liq / \tau av$
Elevation 730	0.79	0.13	0.42	3.18
Elevation 705	2.53	0.66	1.35	2.03
Elevation 680	4.26	1.07	2.27	2.11

* Grade elevation 736 feet.

TABLE 2.5-50
SUMMARY OF LIQUEFACTION ANALYSES
FOR NEW MADRID TYPE EARTHQUAKE

ELEVATION* (FEET)	EFFECTIVE OVERBURDEN PRESSURE $\sigma_o\ (kips/ft^2)$	AVERAGE CYCLIC SHEAR STRESS FOR 30 CYCLES $\tau_{av}\ (kips/ft^2)$	AVERAGE CYCLIC SHEAR STRESS CAUSING LIQUE-FACTION IN 30 CYCLES $\tau_{liq}\ (kips/ft^2)$	FACTOR OF SAFETY WITH RESPECT TO INITIAL LIQUEFACTION τ_{liq}/τ_{av}
Elevation 730	.79	.07	.22	3.35
Elevation 705	2.53	.33	.71	2.14
Elevation 680	4.26	.54	1.19	2.22

* Grade elevation 736 feet.

TABLE 2.5-51 HAS BEEN INTENTIONALLY DELETED.

TABLE 2.5-52
AVERAGE SOIL PROPERTIES OF BORROW MATERIALS FOR MAIN DAM

SOIL PROPERTIES	
Soil Type	Brown and gray silty clay with trace to some sand and trace fine gravel.
	AVERAGE
Natural Moisture Content (%)	12.2
Liquid Limit (%)	23.6
Plasticity Index (%)	9.5
Compaction Test (ASTM D698)	
Maximum Dry Density (lb/ft^3)	124.1
Optimum Moisture Content (%)	11.7
Shear Strength (Remolded Samples):	
Total:*	
Cohesion (lb/ft^2)	1300
Angle of Internal Friction (degrees)	0
Effectiveness:	
Cohesion (lb/ft^2)	200
Angle of Internal Friction (degrees)	33
Permeability* (cm/sec)	$2x10^{-8}$

* Shear Strength and Permeability correspond to samples compacted to a dry density of 90% of the maximum density determined by the AASHO T-180 Method of Compaction, latest revision.

TABLE 2.5-53
SOIL PARAMETERS FOR STATIC ANALYSIS OF NATURAL SLOPE

	LOESS	WISCONINAN GLACIAL TILL OF WEDRON FORMATION	INTER-GLACIAL DEPOSITS	INTER-GLACIAL SAND DEPOSITS	ILLINOIAN GLACIAL TILL OF ALTERED GLASFORD FORMATION	LACUSTRINE DEPOSITS	PRE-ILLINOIAN DEPOSITS
Density (lb/ft^3)	120.0	137.0	131.0	125.0	150.0	134.0	145.0
Coefficient of Earth Pressure at Rest	0.5	1.2	1.2	1.0	1.0	0.6	0.7
Poisson's Ratio	0.35	0.40	0.40	0.40	0.40	0.40	0.40
Modulus of Elasticity (lb/ft^2)	2.0×10^5	2.3×10^5	2.3×10^5	2.0×10^5	4.0×10^5	4.5×10^5	4.5×10^5
Cohesion (lb/ft^2)	0	600	600	0	0	0	1400
Angle of Internal Friction (deg.)	20	30	30	38	47	34	42

TABLE 2.5-54
SUMMARY OF LIQUEFACTION ANALYSES

BORING NUMBER	ELEVATION (feet)	DESCRIPTION OF SAND	N-VALUE (blows/ft)	RELATIVE DENSITY(1) (%)	τ_{avg}[2] (lb/ft²)	τ_{liq}[3] (lb/ft²)	FACTOR OF SAFETY
H-28	671	Gray silty fine to coarse sand, some gravel (SM)	15	80	266	200	0.75
P-5	663	Gray fine to coarse sand, some gravel (SP)	67	90	338	402	1.19
P-8	665	Gray medium to coarse sand with gravel (SP)	25	95	225	246	1.09
P-12	662	Gray fine to coarse sand, some gravel (SP)	23	91	282	286	1.01
H-32	673	Gray fine to coarse sand, some gravel, trace silt (SP)	10	68	210	121	0.57
H-33	678	Brown silty fine to coarse sand, trace clay and gravel (SW)	5	52	157	69	0.44
H-33	676	Brown fine silty sand (SM)	3	48	196	80	0.41
H-33	667	Dark gray fine sand, some silt (SP)	12	66	371	235	0.63

(1) Relative density is obtained from the N-value and effective overburden pressure using relationship given by Gibbs and Holtz.

(2) Average cyclic shear stress for 10 cycles.

(3) Average cyclic shear stress causing liquefaction in 10 cycles. The mean grain size D_{50} of all sands was assumed equal to 0.2 mm, based on available grain size analyses and soil description.

TABLE 2.5-55
SOIL PARAMETERS FOR STATIC ANALYSIS OF SUBMERGED DIKE

	RECOMPACTED WISCONSINAN GLACIAL TILL OF WEDRON FORMATION TYPE A MATERIAL	ILLINOIAN GLACIAL TILL OF ALTERED GLASFORD FORMATION	LACUSTRINE DEPOSITS	PRE-ILLINOIAN DEPOSITS
Density (lb/ft^3)	135.0	150.0	134.0	145.0
Poisson's Ratio	0.40	0.35	0.35	0.35
Modulus of Elasticity (lb/ft^2)	2.0×10^5	4.0×10^5	4.5×10^5	4.5×10^5
Cohesion (lb/ft^2)	400	0	0	1400
Angle of Internal Friction (deg.)	29	47	34	42

TABLE 2.5-56

SUMMARY OF LOCAL FACTORS OF SAFETY IN SUBMERGED DIKE ELEMENTS

ELEMENT NUMBER	CRITICAL NUMBER OF CYCLES N_c	AVERAGE CYCLIC SHEAR STRESS FOR N_c CYCLES τ_{avg} (lb/ft²)	AVERAGE CYCLIC SHEAR STRESS CAUSING 5% STRAIN IN N_c CYCLES τ_{fail} (lb/ft²)	MINIMUM FACTOR OF SAFETY WITH RESPECT TO 5% STRAIN FOR N_c CYCLES τ_{fail}/τ_{avg}
1	5	53	270	5.09
2	5	62	270	4.35
4	5	81	270	3.33
6	5	92	270	2.93
11	6	89	260	2.92
12	6	106	370	3.49
14	6	147	370	2.52
15	6	161	370	2.30
16	6	168	370	2.20
21	6	92	260	2.83
22	6	147	370	2.52
23	4	209	570	2.73
24	5	234	530	2.26
25	6	243	500	2.06
26	6	253	500	1.98
33	4	269	570	2.12
35	6	325	680	2.09
36	6	334	680	2.04
43	8	311	470	1.51
45	7	245	490	2.41
47	6	371	760	2.05
49	6	418	940	2.25
50	6	432	940	2.18
61	6	512	760	1.48
64	6	574	940	1.64
66	6	589	940	1.60
68	3	686	1160	1.69
81	6	798	940	1.18
85	6	859	940	1.09
87	2	1189	1440	1.21
89	3	963	1320	1.37

TABLE 2.5-57

SUMMARY OF LOCAL FACTORS OF SAFETY IN THE NATURAL SLOPE (3.5 HORIZONTAL TO 1 VERTICAL) ELEMENTS

ELEMENT NUMBER	CRITICAL NUMBER OF CYCLES N_c	AVERAGE CYCLIC SHEAR STRESS FOR N_c CYCLES τ_{avg} (lb/ft²)	AVERAGE CYCLIC SHEAR STRESS CAUSING 5% STRAIN IN N_c CYCLES τ_{fail} (lb/ft²)	MINIMUM FACTOR OF SAFETY WITH RESPECT TO 5% STRAIN FOR N_c CYCLES τ_{fail}/τ_{avg}
8	6	177	520	2.94
9	6	196	760	3.88
16	6	323	520	1.61
17	6	322	940	2.92
19	5	352	1000	2.84
21	6	372	940	2.53
22	6	383	940	2.45
26	6	418	940	2.25
36	6	472	940	1.99
39	5	616	1180	1.92
42	5	677	1180	1.74
47	6	497	760	1.53
59	6	495	760	1.54
62	4	856	1240	1.45
65	5	908	1180	1.30
68	5	939	1180	1.26
72	6	461	760	1.65
75	5	870	1180	1.36
87	6	713	1095	1.54
88	6	797	1362	1.71
89	6	922	1629	1.77
90	6	972	2124	2.19
91	6	1063	2391	2.25
92	6	1092	2832	2.59
93	6	1088	2832	2.60
94	6	1068	2832	2.65
95	6	1036	2832	2.73
99	6	303	520	1.72
101	5	662	1000	1.51
121	5	769	1000	1.30

TABLE 2.5-57 (Cont'd)

(Q&R 241.16)

LOCAL FACTORS OF SAFETY IN THE NATURAL SLOPE

ELEMENT NUMBER	CRITICAL NUMBER OF CYCLES N_c	AVERAGE CYCLIC SHEAR STRESS FOR N_c CYCLES avg (lb/ft²)	AVERAGE CYCLIC SHEAR STRESS CAUSING 5% STRAIN IN N_c CYCLES fail (lb/ft²)	MINIMUM FACTOR OF SAFETY WITH RESPECT TO 5% STRAIN FOR N_c CYCLES fail/avg
98	8	366	495	1.35
100	5	531	1290	2.43
101	5	715	1500	2.10
102	5	923	1545	1.68
103	5	1041	1710	1.65
104	5	1138	1787	1.57
105	5	1256	1823	1.46
106	5	1326	1844	1.40
107	5	1386	1844	1.33
108	5	1413	1844	1.31
109	5	1331	1844	1.39
114	9	64	122	1.91
115	7	70	137	1.96
116	9	58	122	2.10
117	9	49	122	2.49
118	7	790	945	1.20
119	6	572	1155	2.02
120	5	675	1380	2.04
122	5	984	1562	1.59
123	5	1116	1727	1.55
124	5	1225	1806	1.47
125	5	1329	1832	1.38
126	5	1433	1865	1.30
127	5	1498	1868	1.25
128	5	1549	1868	1.21
129	5	1627	1868	1.15

The factors of safety for the requested elements are given in the preceding table. The minimum factors of safety for sand elements were obtained by comparing the shear stresses required to cause single-amplitude shear strain of 5% (see Figure 2.5-413) with the equivalent shear stresses induced by the earthquake. It can be seen from the following table that the 5% strain occurs before the initial liquefaction starts. Therefore, the criterion of 5% strain is conservative, and is assures that the sand elements will not liquefy during the earthquake (Q&R 241.16).

TABLE 2.5-57

(Q&R 241.16)

RESULTS OF STRESS-CONTROLLED CYCLIC TRIAXIAL (LIQUEFACTION) TESTS

BORING OR TEST PIT NUMBER	DEPTH OF SAMPLE ft.	SOIL DESCRIPTION	DEGREE OF COMPACTION	MOLDING MOISTURE CONTENT, %	PRINCIPAL CONSOLIDATION STRESS RATIO kC	LATERAL CONSOLIDATION PRESSURE 3C, psf	CYCLE STRESS RATIO V/2 3c	SKEMPTON'S PORE PRESSURE, PARAMETER, B	5% STRAIN[1]	10% STRAIN[1]	20% STRAIN[1]	INITIAL LIQUE-FACTION[2]
Interglacial Granular Soils:												
TP-3	9-11	Bluish gray fine to medium sand with traces of silt and fine gravel	75% (Relative)	7.9	1.0	2000	0.21	0.97	66	70	92	92
				7.6	1.0	2000	0.39	1.0	8	11	20	9
				7.4	1.0	2000	0.61	0.95	3	5	10	5
H-23	49	Brown silty fine sand	--[3]	16.9[5]	1.0	2000	0.38	0.99	25	90	200	200
H-31	14	Gray gravelly fine to coarse sand with some silt	--[4]	8.6[5]	1.0	2000	0.37	0.96	8	28	56	8

1 Double Amplitude Axial Strain.
2 Initial liquefaction is defined as when the increase in pore pressure is equal to the effective confining pressure.
3 "Undisturbed" sample taken during the boring operations, dry density = 112.8 pcf.
4 "Undisturbed" sample taken during the boring operations, dry density = 125.6 pcf.
5 Initial (in-situ) moisture content.

TABLE 2.5-58

SUMMARY OF LOCAL FACTORS OF SAFETY IN SUBMERGED DIKE ELEMENTS DUE TO A NEW MADRID TYPE EARTHQUAKE

ELEMENT NUMBER	CRITICAL NUMBER OF CYCLES N_c	AVERAGE CYCLIC SHEAR STRESS FOR N_c CYCLES τ_{avg} (lb/ft²)	AVERAGE CYCLIC SHEAR STRESS CAUSING 5% STRAIN IN N_c CYCLES τ_{fail} (lb/ft²)	MINIMUM FACTOR OF SAFETY WITH RESPECT TO 5% STRAIN FOR N_c CYCLES τ_{fail}/τ_{avg}
1	20	11.6	175	15.09
2	20	19.3	175	9.07
4	20	29.1	175	6.01
6	20	32.2	175	5.44
11	30	27.6	80	2.90
12	10	53.5	313	5.85
14	10	71.1	313	4.40
15	10	75.9	313	4.12
16	10	75.6	313	4.14
21	30	31.7	80	2.52
22	10	78.1	313	4.01
23	10	97.6	432	4.43
24	10	111.1	432	3.89
25	10	117.7	432	3.67
26	10	117.5	432	3.68
33	10	129.8	432	3.33
35	10	164.9	610	3.70
36	10	164.7	610	3.70
43	20	57.3	320	5.59
45	20	98.3	320	3.26
47	13	185.7	610.8	3.29
49	13	207.4	763.2	3.68
50	13	205.4	763.2	3.72
61	13	203.2	610.8	3.01
64	10	281.2	820	2.92
66	10	306.8	820	2.67
68	10	312.0	820	2.63
81	10	426.1	820	1.92
85	10	453.9	820	1.81
87	10	461.2	1000	2.17
89	10	454.1	1000	2.20

TABLE 2.5-59

SUMMARY OF LOCAL FACTORS OF SAFETY IN THE NATURAL SLOPE (3.5 HORIZONTAL TO 1 VERTICAL) ELEMENTS DUE TO A NEW MADRID TYPE EARTHQUAKE

ELEMENT NUMBER	CRITICAL NUMBER OF CYCLES N_c	AVERAGE CYCLIC SHEAR STRESS FOR N_c CYCLES τ avg (lb/ft²)	AVERAGE CYCLIC SHEAR STRESS CAUSING 5% STRAIN IN N_c CYCLES τ fail (lb/ft²)	MINIMUM FACTOR OF SAFETY WITH RESPECT TO 5% STRAIN FOR N_c CYCLES τ fail/ τ avg
8	20	97.7	320	3.28
9	10	128.7	660	5.13
16	20	163.9	320	1.95
17	6	257.8	950	3.69
19	6	261.4	950	3.63
21	6	269.1	950	3.53
22	7	235.1	910	3.87
26	6	345.0	950	2.75
36	6	398.1	950	2.39
39	4	521.8	1240	2.38
42	4	541.2	1240	2.29
47	6	426.6	770	1.81
59	6	432.2	770	1.78
62	4	709.2	1240	1.75
65	4	770.5	1240	1.61
68	4	760.7	1240	1.63
72	6	407.3	770	1.89
75	4	767.9	1240	1.62
87	20	394.0	648	1.65
88	20	433.1	806	1.86
89	18	521.2	1013.0	1.94
90	18	545.7	1320.7	2.42
91	18	592.5	1486.7	2.51
92	18	604.6	1760.9	2.91
93	18	599.6	1760.9	2.94
94	18	587.5	1760.9	3.00
95	18	572.6	1760.9	3.08
99	25	151.2	287	1.90
101	6	539.9	950	1.76
121	5	663.2	1000	1.51

TABLE 2.5-60
ATTERBERG LIMITS DATA
ULTIMATE HEAT SINK BORINGS

BORING NUMBER	ELEVATION (ft-in.)	SOIL TYPE	GEOLOGIC UNIT	PLASTIC LIMIT (%)	PLASTICITY INDEX (%)
H-2	663.9	SM/ML	Salt Creek Alluvium	20.4	20.4
H-2	627.4	ML	Illinoian Glacial Till	12.8	7.3
H-3	624.6	ML	Illinoian Glacial Till	12.2	9.3
H-4	657.6	ML	Illinoian Glacial Till	11.9	6.1
H-4	622.6	ML	Illinoian Glacial Till	11.8	7.6
H-4	602.6	ML	Illinoian Glacial Till	11.6	8.0
H-5	653.6	ML	Illinoian Glacial Till	11.6	6.5
H-5	608.6	ML	Illinoian Glacial Till	11.4	8.6
H-6	653.3	ML	Illinoian Glacial Till	11.5	7.6
H-6	583.3	ML	Illinoian Glacial Till	11.5	5.1
H-6	578.3	CL	Illinoian Glacial Till	13.6	12.0
H-7	649.5	ML	Illinoian Glacial Till	10.0	7.7
H-7	630.0	ML	Illinoian Glacial Till	11.8	2.6
H-10	623.2	ML	Illinoian Glacial Till	12.2	8.1
H-11	598.1	ML	Illinoian Glacial Till	11.4	8.3
H-12	635.6	ML	Illinoian Glacial Till	10.9	4.6
H-15	711.3	ML	Wisconsinan Glacial Till	12.0	1.6

TABLE 2.5-60 (Cont'd)

BORING NUMBER	ELEVATION (ft-in.)	SOIL TYPE	GEOLOGIC UNIT	PLASTIC LIMIT (%)	PLASTICITY INDEX (%)
H-18	645.5	ML	Illinoian Glacial Till	11.5	6.9
H-20	701.8	ML	Wisconsinan Glacial Till	13.5	11.6
H-25	638.7	ML	Illinoian Glacial Till	11.3	5.6
H-31	689.1	ML	Wisconsinan Glacial Till	13.1	16.4
H-31	680.1	CL	Interglacial Zone	13.8	23.5
H-32	555.1	ML	Pre-Illinoian Glacial Till	12.9	13.2
H-33	674.8	SM	Salt Creek Alluvium	N.P.	N.P.
H-36	541.2	ML	Pre-Illinoian Glacial Till	12.8	11.9
H-3	654.6	ML	Illinoian Glacial Till	11.5	6.5
H-5	634.1	ML	Illinoian Glacial Till	10.8	5.5
H-8	658.3	SM	Salt Creek Alluvium	N.P.	N.P.
H-10	643.7	ML	Illinoian Glacial Till	10.9	4.2
H-12	671.1	ML	Salt Creek Alluvium	22.6	20.0
H-12	663.6	ML	Illinoian Glacial Till	12.4	5.4
H-16	662.3	ML	Illinoian Glacial Till	13.2	6.0
H-24	670.7	SM	Salt Creek Alluvium	N.P.	N.P.
H-24	627.7	ML	Illinoian Glacial Till	12.3	6.1
H-25	658.2	ML	Salt Creek Alluvium	11.3	5.8

TABLE 2.5-60 (Cont'd)

BORING NUMBER	ELEVATION (ft-in.)	SOIL TYPE	GEOLOGIC UNIT	PLASTIC LIMIT (%)	PLASTICITY INDEX (%)
H-29	682.7	ML	Illinoian Glacial Till	14.8	11.8
H-32	620.1	ML	Illinoian Glacial Till	11.9	8.6
H-33	662.8	ML	Illinoian Glacial Till	N.P.	N.P.
H-34	652.6	ML	Illinoian Glacial Till	11.0	5.3
H-34	632.6	ML	Illinoian Glacial Till	11.8	7.8
H-35	632.8	ML	Illinoian Glacial Till	10.8	6.0
H-37	689.9	ML	Interglacial Soil	20.1	22.6

TABLE 2.5-61
MODIFIED MERCALLI INTENSITY (DAMAGE) SCALE OF 1931

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 many people do not recognize it 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 make creaking sound. Sensation like heavy truck striking building. Standing motorcars rock 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 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 runs 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)

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 remain standing. Bridges destroyed. Broad fissures in ground. Underground pipelines completely out of service. Earth slumps and land slips 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 2.5-62
SUMMARY OF THE CONSOLIDATION TEST DATA

BORING NUMBER	ELEVATION (feet)	GEOLOGICAL MEMBER	MOISTURE CONTENT (percent)	DRY DENSITY (pcf)	INITIAL[1] VOID RATIO	SAMPLING METHOD	ATTERBERG LIMITS (percent) PLASTIC LIMIT	ATTERBERG LIMITS (percent) PLASTICITY INDEX	COMPRESSION INDEX	RE-COMPRESSION INDEX	PRE-CONSOLIDATION[2] PRESSURE (psf)
P-14	683.8	Interglacial Till	16.2	115	0.41	U	12.5	12.5	0.12	0.03	4,100
P-15	711.8	Wisconsinan Till	12.8	116	0.40	U	14.5	8.5	0.14	0.025	5,700
P-18	683.7	Illinoian Till	17.7	104	0.56	U	NA	NA	0.19	0.03	10,500
P-20	688.3	Interglacial Till	10.9	122	0.33	U	14.4	7.2	0.12	0.023	8,000
P-21	695.7	Interglacial Till	18.4	114	0.42	U	16.6	4.4	0.07	0.011	>5,000
P-22	728.5	Loess	22.2	97	0.68	U	22.8	17.3	0.25	0.044	2,100
	699.5	Interglacial Till	36.8	74	1.19	U	NA	NA	0.36	0.030	4,500
	689.5	Interglacial Till	12.8	119	0.37	U	12.4	5.0	0.085	0.014	5,000
P-26	692.1	Interglacial Till	14.6	120	0.35	U	9.8	11.6	0.11	0.024	4,500
P-27	728.4	Wisconsinan Till	11.2	126	0.29	U	10.4	6.8	0.083	0.016	4,000
	703.4	Interglacial Till	17.5	109	0.49	U	12.4	11.5	0.13	0.022	>4,000
P-29	706.5	Wisconsinan Till	7.5	121	0.34	U	16.8	9.2	0.11	0.014	3,800
	667.0	Illinoian Till	17.7	110	0.47	P	NA	NA	0.094	0.017	17,000
	627.0	Illinoian Till	9.9	134	0.21	P	11.0	5.7	0.089	0.016	22,000
	617.0	Illinoian Till	10.1	136	0.19	U	NA	NA	0.11	0.017	19,500
	577.0	Illinoian Till	7.6	141	0.15	P	11.7	35.0	0.082	0.008	22,000
	522.0	Pre-Illinoian Till	14.9	116	0.40	P	NA	NA	0.15	0.036	23,000
	517.0	Pre-Illinoian Till	16.9	116	0.40	P	15.7	9.7	0.16	0.036	14,000

TABLE 2.5-62(Cont'd)

BORING NUMBER	ELEVATION (feet)	GEOLOGICAL MEMBER	MOISTURE CONTENT (percent)	DRY DENSITY (pcf)	INITIAL[1] VOID RATIO	SAMPLING METHOD	ATTERBERG LIMITS (percent) PLASTIC LIMIT	ATTERBERG LIMITS (percent) PLASTICITY INDEX	COMPRESSION INDEX	RE-COMPRESSION INDEX	PRE-CONSOLIDATION[2] PRESSURE (psf)
P-32	667.4	Illinoian Till	8.4	136	0.19	C	11.3	10.8	--	0.009	18,000
	644.2	Illinoian Till	6.7	138	0.18	C	11.3	6.6	0.13	0.012	20,000
	627.9	Illinoian Till	6.9	140	0.16	C	10.9	8.9	0.062	0.010	17,000
	585.0	Illinoian Till	8.5	136	0.19	C	10.6	7.8	0.11	0.014	25,000
P-33	675.6	Illinoian Till	7.2	139	0.17	U	NA	NA	0.053	0.013	>20,000
P-36	639.2	Illinoian Till	6.5	140	0.16	C	10.7	6.5	0.059	0.011	18,000
	613.7	Illinoian Till	8.6	135	0.20	C	11.9	9.0	0.13	0.014	20,000
	599.2	Illinoian Till	7.9	138	0.18	C	11.8	7.0	0.090	0.009	25,000
	559.2	Pre-Illinoian Till	6.8	137	0.18	C	NA	NA	0.094	0.010	21,000
	538.7	Pre-Illinoian Till	16.3	116	0.4	C	17.8	25.5	0.24	0.020	23,000
P-38	655.6	Illinoian Till	7.7	138	0.18	C	9.2	6.0	0.11	0.007	16,000
	638.4	Illinoian Till	6.4	139	0.17	C	9.7	5.8	0.095	0.009	17,000
	634.9	Illinoian Till	6.9	139	0.17	C	NA	NA	0.053	0.010	>20,000
	616.9	Illinoian Till	10.1	132	0.23	C	NA	NA	0.18	0.015	21,000
	598.7	Illinoian Till	8.4	138	0.18	C	9.8	6.6	0.11	0.009	17,000
	569.9	Lacustrine	11.7	126	0.29	U	NA	NA	0.11	0.020	24,000
	567.6	Lacustrine	10.6	129	0.26	C	10.2	7.5	0.12	0.013	22,000
	531.5	Pre-Illinoian Till	16.0	116	0.40	C	17.5	21.0	0.17	0.013	23,000
P-41	714.7	Wisconsinan Till	11.1	128	0.27	C	NA	NA	0.034	0.0053	5,000
	679.7	Illinoian Till	7.5	138	0.18	C	NA	NA	0.031	0.014	15,500

TABLE 2.5-62(Cont'd)

LEGEND

NA - Test result not available

C - 4-inch core sampler

P - Pitcher sampler

U - Dames & Moore "U" sampler

Notes: 1. Initial void ratio computed by assuming specific gravity equal to 2.6.

2. Preconsolidation pressures determined by using A. Casagrande approach.

TABLE 2.5-63
BEARING CAPACITY AND FACTOR OF SAFETY

FOUNDATION AREA	SEISMIC CATEGORY I STRUCTURE	FOUNDATION ELEVATION (feet)	GROSS APPLIED STATIC FOUNDATION PRESSURE (ksf)	NET STATIC FOUNDATION PRESSURE* (ksf)	ULTIMATE BEARING CAPACITY (ksf)	FACTOR OF SAFETY
Containment	X	702.0	6.5	4.8	90.1	18.8
Fuel Building	X	702.0	6.5	4.8	110.4	23.0
Auxiliary	X	697.5	6.5	4.5	91.8	20.4
Radwaste, Machine Shop, and Off-Gas Building	X	692.0	4.8	2.4	121.2	50.5
Service Building		732.0	1.5	1.5	51.2	34.1
Diesel Generator and HVAC	X	702.0	4.7	3.0	79.8	26.6
Control	X	692	4.7	2.3	84.1	36.6
Turbine and Heater Bay		702.0	5.7	4.0	120.3	30.1
Circulating Water Screen House	X	653.0	3.2	2.9	244.0	84.1
Ultimate Heat Sink Outlet Structure	X	669.0	0.6	.48	14.5	30.2

* The net static foundation pressure equals the gross applied static foundation pressure minus the hydrostatic uplift pressure.

TABLE 2.5-64
TYPICAL EXAMPLE OF CALCULATION OF FACTOR OF SAFETY

CYCLE	STRESSES (lb/ft^2) +ve	STRESSES (lb/ft^2) -ve	STRESSES IN DESCENDING ORDER (lb/ft^2) +ve	STRESSES IN DESCENDING ORDER (lb/ft^2) -ve	AVERAGE STRESS (lb/ft^2)	CUMULATIVE AVERAGE STRESS (lb/ft^2)	τ FAILURE (lb/ft^2)	FACTOR OF SAFETY
1	113	204	169	218	194	194	620	3.20
2	132	218	150	212	181	188	520	2.77
3	149	162	149	204	177	184	470	2.55
4	150	130	132	162	147	175	430	2.45
5	80	106	118	130	124	165	400	2.42
6	169	212	113	130	122	158	370	2.34
7	48	65	111	106	109	151	360	2.38
8	111	130	106	106	106	145	350	2.40
9	118	28	106	103	105	141	340	2.41
10	106	103	85	77	81	135	330	2.44
11	30	38	80	65	73	129	325	2.52
12	71	77	71	53	62	123	320	2.60
13	14	106	52	52	52	118	315	2.67
14	52	31	48	38	43	113	310	2.74
15	106	53	30	31	31	107	305	2.85
16	85	52	14	28	21	102	300	2.94

TABLE 2.5-65
SEISMIC SOURCE PARAMETERS

	SOURCE	ACTIVITY RATE (MEAN NUMBER OF EARTHQUAKES/YEAR)	HISTORICAL MAXIMUM INTENSITY	UPPER BOUND INTENSITY
1.	Illinois Basin Seismogenic Region	0.20	VII	VIII
2.	Ste. Genevieve Seismogenic Region	0.02	VI	VII
3.	St. Francois Mountains Seismogenic Region	0.03	VI-VII	VII-VIII
4.	Chester Dupo Seismogenic Region	0.27	VII	VIII
5.	Wabash Valley Seismogenic Region	0.17	VII	VIII
6.	Western Kentucky Fault Zone Seismotectonic Region	0.03	V	VI
7.	Iowa-Minnesota Seismogenic Region	0.01	V	VI
8.	Missouri Random Seismogenic Region	0.02	V	VI
9.	Michigan Basin Seismogenic Region	0.02	VI	VII
10.	Eastern Interior Arch System Seismogenic Region	0.04	VII	VIII
11.	Anna Seismogenic Region	0.25	VII-VIII	VIII-IX
12.	New Madrid Seismogenic Region	0.33	XII	XII

TABLE 2.5-66
RECOMPRESSION INDEX FOR SETTLEMENT ANALYSIS

	$C_r/(1+e_o)$	
ELEVATIONS	RELOADING BELOW P_o AND REBOUND	RELOADING ABOVE P_o BUT BELOW P_C
660 to 680 ft	0.005	0.0110
627 to 660 ft	0.005	0.0083
614 to 627 ft	0.005	0.0142
575 to 615 ft	0.005	0.0091
560 to 575 ft	0.005	0.0165
540 to 560 ft	0.005	0.0117
522 to 540 ft	0.005	0.0171
500 to 522 ft	0.005	0.0257

NOTES:

1. P_o = In situ overburden pressure
2. P_C = Overconsolidation pressure
3. C_r = Recompression index
4. e_o = Initial void ratio

TABLE 2.5-67
COMPARISON OF CALCULATED AND MEASURED SETTLEMENTS

BUILDING	SETTLEMENT MONUMENT*	CALCULATED FINAL SETTLEMENT (in.)	MEASURED*** SETTLEMENT (in.)	DATE OF FIRST MEASUREMENT	DATE OF LAST MEASUREMENT
Containment	C1A**	1.72	-0.096	Jan. 1981	Sept. 1983
	C2	1.72	0.264	May 1978	Jan. 1984
	C3	1.64	0.108	May 1978	Sept. 1983
	C4	1.79	0.504	May 1978	May 1982
	C5	1.75	0.384	May 1978	Jan. 1984
	C7	1.59	0.384	May 1978	Jan. 1984
	C8A**	1.72	-0.048	Jan. 1981	Jan. 1984
	C9	1.59	0.420	May 1978	May 1982
	C10	1.61	0.360	May 1978	Jan. 1984
	C11	1.37	0.492	May 1978	Jan. 1982
Turbine	T1A**	1.45	-0.048	Sept. 1980	Jan. 1984
	T2	1.44	0.228	May 1978	Jan. 1984
	T3A**	1.40	0.096	Sept. 1980	Jan. 1984
	T4A**	1.36	-0.060	Nov. 1980	Jan. 1984
Diesel Generator & Control	DIAB**	1.22	-0.036	Nov. 1980	Jan. 1984
	D2	1.47	0.360	May 1978	Jan. 1984
	D3	1.18	0.252	May 1978	Jan. 1984
	D4A**	1.42	-0.072	Mar. 1981	Jan. 1984
	D5	1.11	0.240	May 1978	Jan. 1984
	D6	1.01	0.444	May 1978	May 1983
	D7	1.26	0.180	May 1978	Jan. 1984
Radwaste Off-Gas & Machine Shop	R1A**	1.15	-0.156	Mar. 1980	Jan. 1984
	R2B**	1.05	-0.060	Nov. 1980	Jan. 1984
	R3	1.20	0.024	May 1978	Jan. 1984
	R4A**	0.97	-0.168	May 1980	Jan. 1984
	R5A**	1.07	-0.168	Sept. 1981	Jan. 1984

* See Figure 2.5-382 for locations of settlement monuments.

** D1 and D1A were replaced by D1B, R1 by R1A, R2 and R2A by R2B, R4 and R4A, R5 by R5A, C1 by C1A, C8 by C8A, T1 by T1A, T3 by T3A, T4 by T4A, and D4 by D4A.

*** Representing difference between first and last readings. Negative signs indicate heave.

TABLE 2.5-68
(Q&R 241.16)
RESULTS OF STRESS-CONTROLLED CYCLIC TRIAXIAL (LIQUEFACTION) TESTS

BORING OR TEST PIT NUMBER	DEPTH OF SAMPLE ft.	SOIL DESCRIPTION	DEGREE OF COMPACTION	MOLDING MOISTURE CONTENT, %	PRINCIPAL CONSOLIDATION STRESS RATIO Kc	LATERAL CONSOLIDATION PRESSURE 3c, psf	CYCLE STRESS RATIO v/2 3c	SKEMPTON'S PORE PRESSURE, PARAMETER, B	NUMBER OF CYCLES TO CAUSE			
									5% STRAIN[1]	10% STRAIN[1]	20% STRAIN[1]	INITIAL LIQUEF-ACTION[2]
Interglacial Granular Soils:												
TP-3	9-11	Bluish gray fine to medium sand with traces of silt and fine gravel	75% (Relative)	7.9 7.6 7.4	1.0 1.0 1.0	2000 2000 2000	0.21 0.39 0.61	0.97 1.0 0.95	66 8 3	70 11 5	92 20 10	92 9 5
H-23	49	Brown silty fine sand	-3	16.9[5]	1.0	2000	0.38	0.99	25	90	200	200
H-31	14	Gray gravelly fine to coarse sand with some silt	-4	8.6[5]	1.0	2000	0.37	0.96	8	28	56	8

1 Double Amplitude Axial Strain.

2 Initial liquefaction is defined as when the increase in pore pressure is equal to the effective confining pressure.

3 "Undisturbed" sample taken during the boring operations, dry density = 112.8 pcf.

4 "Undisturbed" sample taken during the boring operations, dry density = 125.6 pcf.

5 Initial (in-situ) moisture content.

ATTACHMENT A2.5

ULTIMATE HEAT SINK LIQUEFACTION ANALYSIS

ATTACHMENT A2.5 - ULTIMATE HEAT SINK LIQUEFACTION ANALYSIS

A2.5 ULTIMATE HEAT SINK LIQUEFACTION ANALYSIS

A2.5.1 Summary

Four failure modes were postulated which could infringe upon the capability of the Clinton Power Station (CPS) proposed ultimate heat sink (UHS) to perform its function. These four postulated failure modes are:

a. loss of cooling water inventory due to its displacement by alluvial flow slides into the UHS;

b. loss of the service water system due to blockage of the service water pump intakes from flow blocking or entering the intake structure;

c. loss of the UHS circulation pattern due to local slides producing dikes or dams across the circulation channel; and

d. loss of UHS water because the UHS dam or its flanks are breached by the combination of seismic loadings, liquefaction, and washout.

Field exploration programs, laboratory testing programs, evaluation of soil instability through empirical and analytical methods, and evaluation of the consequences of postulated soil instability have led to the conclusion that the capability of the ultimate heat sink cannot be compromised by those failure modes.

This attachment summarizes the investigations used to arrive at the above conclusion.

A2.5.2 Ultimate Heat Sink Description

The ultimate heat sink is a submerged pond lying in the bottom of the cooling lake (Lake Clinton). A compacted earth dam lying across the lower portion of the North Fork stream valley retains the pond, and the required storage capacity was developed by excavating the valley alluvium to provide 1067 acre-feet storage capacity. In the event of loss of the cooling lake, recirculation within the pond will be guided by a baffle dike to gain the effective surface area required for heat dissipation.

The dam and the baffle dike were constructed from cohesive soils available in the site area. These earth structures were founded on the Illinoian glacial till of the unaltered Glasford Formation which underlies the alluvial deposits presently in the stream valley.

Figure A2.5-1 presents a plan of the ultimate heat sink.

A2.5.3 Design Bases

The ultimate heat sink will provide sufficient water volume and cooling capability for the station for 30 days with no water makeup. Subsection 9.2.5.3 presents the safety evaluation for the UHS. The ultimate heat sink will also provide a minimum of 900,000 gallons of water for fire protection, if required.

In addition to the storage requirements for cooling purposes and fire water supply, storage capacity is provided to accept some sedimentation. Sediment accumulation within the heat sink will be periodically measured, and in the event that accumulation exceeds the capacity provided for sediment storage, dredging will be performed.

Significant design bases used in the determination of the minimum UHS volume include the following:

a. sediment inflow from liquefaction - 221 acre-feet;

b. required fire water storage capacity - 3 acre-feet;

c. minimum cooling capacity requirement of UHS to meet 95° F shutdown service water inlet temperature - 590 acre-feet;

d. an uninterrupted flow path must be maintained at all times between the UHS inlet structure and the shutdown service water pumps; and

e. maximum sedimentation due to a 100-year flood - 35 acre-feet.

A2.5.4 Geotechnical Investigation

A2.5.4.1 Introduction

An evaluation of the soil slopes adjacent to the ultimate heat sink was conducted to determine the horizontal and vertical distribuition, relative denseness, and geometry of these potentially liquefiable alluvial soils.

The field investigation portion of the program consisted of fourteen BH-series borings and twelve PH-series auger probes. The location of the borings and the probes are shown in Figure A2.5-2. To determine the consistency of the subsurface strata, borings were drilled with truck mounted rotary wash equipment using 4-inch continuous-flight augers, with samples being extracted by utilizing a standard 2-inch split-spoon sampler.

A graphic representation of soils encountered in the borings, including standard penetration test data and sampling information, is presented in Figures A2.5-3 through A2.5-28. The method of classifying the soils is described in Figure 2.5-355. A key to the sample symbols and samplings information presented on the logs of the borings is shown in Figure A2.5-29.

The laboratory portion of the geotechnical investigation consisted of a program to identify the physical characteristics of the soils. Testing of the samples consisted of determining the Atterberg limits, moisture content, particle size, and unit weight determination. The results of these tests are shown in the boring logs presented as Figures A2.5-3 through A2.5-28.

In areas where the alluvium is present, the generalized profile consists of a cohesive deposit underlain by a granular deposit which is in turn underlain by competent till materials of the unaltered Glasford Formation.

A study of the heat sink's periphery reveals that certain sections of the bordering soil deposits may be characterized as possessing common geometrical configuration and material composition. Therefore, the heat sink's periphery has been divided into subsections identified by letters A through F in Figure A2.5-1 and designated as follows:

a. Subsection I - Till Slopes from A to B;

b. Subsection II - Till Slopes with Fringe of Alluvium from B to C;

c. Subsection III - Retaining Fill from C to D;

d. Subsection IV - Northwest Corner Alluvium Pocket from D to E;

e. Subsection V - Outwash Deposits from E to F; and

f. Subsection VI - Upstream Bed Alluvium from F to A.

A2.5.4.2 Subsection I - Till Slopes from A to B

As shown in Figure A2.5-1, the eastern end of the heat sink is bordered by till slopes which were graded to ensure stability under all loading conditions. Both the service pump structure and the SSWS outlet structure lie within this subsection, and the structures were founded on stable Illinoian glacial till of the unaltered Glasford Formation.

The glacial tills which form the slopes are not susceptible to liquefaction, and during construction these slopes were graded to a uniform slope of 5:1 (horizontal to vertical).

A2.5.4.3 Subsection II - Till Slopes with Fringe of Alluvium from B to C

The south side of the heat sink is formed by alluvial soil deposits which lie at the base of natural till slopes which rise above the heat sink. The horizontal extent of the alluvium is shown by the crosshatched area in Figure A2.5-1. The quantity of alluvium postulated to slide into the heat sink from this fringe has been determined by assuming a 30:1 slope (horizontal to vertical), to be approximately 90 acre-feet.

A2.5.4.4 Subsection III - Compacted Earth Retaining Fill from C to D

A compacted earth dam was constructed from cohesive materials not susceptible to liquefaction. Soil borings located in Figure A2.5-2 in the area of the dam have been used to determine the alluvium-Illinoian till interface. Section 11-11 (for plant location, see Figure A2.5-2) in Figure A2.5-30 shows the existing alluvium-till interface; it also shows that existing alluvial material was removed and that the dam was constructed on Illinoian glacial till. The retaining fill spans the valley, and its ends intersect Illinoian glacial till at elevation 675 feet on each end.

No material from Subsection III will move and displace the ultimate heat sink storage volume.

A2.5.4.5 Subsection IV - Northwest Corner Alluvium Pocket from D to E

A pocket of alluvium lies in the northwest corner of the heat sink. Exploratory borings, BH and PH series, were drilled in the area to confirm the contact between the alluvium and the Illinoian till. These borings are located in the plan in Figure A2.5-2. Since this alluvium is potentially susceptible to liquefaction, liquefaction is postulated by assuming a 30:1 slope (horizontal to vertical). It is postulated that 80 acre-feet of material will slide into the heat sink from this section.

A2.5.4.6 Subsection V - Outwash Deposits from E to F

The valley of the North Fork of Salt Creek was formed by melt-water from the Wisconsinan ice sheet (Woodfordian Substage) eroding through Wisconsinan till of the Wedron Formation and down to the Illinoian till of the unaltered Glasford Formation. Subsequent to the downcutting action, outwash material consisting of gravel, sand, and silt was deposited on the Illinoian till within the valley, partially filling it. The outwash material has been identified as part of the Henry Formation and generally varies from clean, well graded sands with some fine gravel to silty sands and silts (Reference 3). The clean sands generally form the basal part of the outwash and the silty sands generally form the upper part.

With the retreat of the ice sheet, the stream eroded the present channel in the outwash material leaving the terrace that is north of the heat sink. The outwash material that forms the terrace is capped in places by colluvial material which has been deposited from late Wisconsinan time to the present. The colluvial material is a silty clay with a trace of sand and fine gravel. The material is generally derived from erosion of the till uplands.

BH series borings and PH series probes used to evaluate the outwash material in the terrace deposits are located in the plan in Figure A2.5-2.

The terrace deposits consist of medium stiff clayey silt material capping the outwash materials. The outwash materials generally consist of two layers. The top layer is composed of approximately 4 feet of loose silt and sand deposits with an average blow count of N = 6. The silty sand is underlain by a layer, approximately 11 feet thick, of well graded, dense sand which directly overlies the glacial till. The outwash materials surface has a relatively flat terrain with an average rise of 10 feet in 900 feet.

Slopes bordering the northern perimeter of the ultimate heat sink were examined by drawing five cross sections, Section 7-7 through 11-11, which are shown in Figures A2.5-31 and A2.5-30, respectively, and are located in Figure A2.5-2. These sections show the subsurface conditions in the area.

In order to establish a conservative final slope for the evaluation of the potential for liquefaction of alluvial soils, the mechanism required to produce significant deformation and movement of the existing slopes must be considered. During the earthquake, before the liquefaction of the alluvium occurs, the soil in the existing slopes is subjected to varying shear stresses which in turn produce excess pore pressures. However, as the pore pressures build up in the granular alluvium, its ability to propagate seismic shear waves would be progressively diminished. Further, when the soil is liquefied prior to any movement, additional development of shear stresses would greatly be reduced. Finally, during the movement of soil after liquefaction there would be a further reduction in the pore pressure buildup through release of pore water.

On this basis, the movement of liquefied soils is restricted to a finite slope dependent on site conditions. This evaluation uses a conservative slope of 30:1 (horizontal to vertical). This slope is based on past case histories (Reference 1) and collaborated by the testimony of J. Greeves of the NRC staff (Reference 2). The flow slide was postulated to determine the final configuration of this area after SSE.

By applying the 30:1 slope criterion, and balancing the soil volumes displaced above the final slope with the volume of the soil filling the original cross section below the final grade, the resultant cross section is established. The initial and final sections which were investigated for this analysis are drawn to natural scale and shown in Figure A2.5-31. The following observations are made in regard to the resultant configurations:

a. Major portions of loose sand likely to liquefy during a SSE (identified in the cross section as SM layer) lies at a slope less than 30:1 (horizontal to vertical) and hence would not slide into the heat sink.

b. Major portion of material carried into the heat sink with the postulated slide consists of plastic, silty clay, which due to its physical characteristics is unlikely to liquefy and flow unless taken for a ride along with underlying liquefied loose silty sand (SM) layer.

On this basis the final cross sections indicated that the changed geometry of the original cut slopes will not impede the circulation of the cooling water. At Section 8-8, the toe of the postulated slope is 280 feet from the baffle dike. Moreover, the increase in the surface area as a result of a slide will somewhat make up for the restricted flow and add to the amount of water circulating in that section.

The total material assumed to flow into the UHS is 50 acre-feet. The quantity is based on the area shown in Section 8-8 which shows the greatest amount of material that will flow in the heat sink in this section.

A2.5.4.7 Subsection VI - Upstream Bed Alluvium

The movement of the upstream bed alluvium is discussed in Reference 1. No movement of material is anticipated into the ultimate heat sink considering the viscous properties of the liquefied soil mass. Under extremely conservative conditions, less than 1 acre-foot is postulated to flow into the UHS. The postulated flow, in the form of a block, is approximately 20 feet into the UHS leaving it more than 1100 feet away from the screen house.

A2.5.5 Conclusions

Four failure modes were postulated which could infringe upon the capability of the CPS proposed ultimate heat sink (UHS) to perform its function. These four postulated failure modes are:

a. (Loss of cooling water inventory due to its displacement by alluvial flow slides into the UHS.) Postulated upper bound slides were used to evaluate potential heat sink capacity losses. The resulting losses are associated with the following perimeter subsection identified by letters A through F in Figure A2.5-1.

Subsection I - Till Slopes from A to B; 0 acre-foot

Subsection II - Till Slopes with Fringe of Alluvium from B to C; 90 acre-feet

Subsection III - Retaining Fill from C to B; 0 acrefoot

Subsection IV - Northwest Corner Alluvium Pocket from D to E; 80 acre-feet

Subsection V - Outwash Deposits from E to F; 50 acre-feet

Subsection VI - Upstream Bed Alluvium from F to A; 1 acre-foot.

The evaluation for displacement of water by alluvium does not take credit for the change in pond configuration which will occur after the postulated slide has taken place. Sloughing of the north and south sides of the pond could, in effect, increase the pond's cooling capability by increasing surface area.

For the ultimate heat sink capacity requirements see USAR Subsections 2.4.11.6 and 9.2.5.

b. Loss of the shutdown service water system due to blockage of the shutdown service water pump intakes cannot occur because unstable soils do not lie at an elevation above the pump intakes in the area of the pump structure. The closest alluvium higher than the pump structure's intakes occurs at point A shown in Figure A2.5-1. This point lies approximately 700 feet from both the pump structure and the baffle dike. The analyses presented in "Potential for Alluvium in North Fork of Salt Creek to Flow into Heat Sink and Prevent Cooling Water Flow" (Reference 1), an unpublished report which has been submitted to the NRC Staff, establishes that the upstream alluvium at point A cannot block flow into the pump intakes.

c. Loss of the UHS circulation pattern due to local slides has been evaluated by considering the final configuration of the postulated slides. Subsection II of the heat sink border is approximately 3600 feet long and along this distance 90 acre-feet is postulated to slide into the sink. The 90 acre-feet of material is uniformally distributed along Subsection II, and its cross-sectional area (perpendicular to the flow path) is approximately one-fifth of the area of the initial channel between the border and the baffle dike. Since material is available to block only one-fifth of the flow path along Subsection II, and since this material lies in space which would be vacated by soil and become heat sink capacity in the event of a slide, blockage of the circulation pattern along Subsection II is not considered to be a credible postulation.

Cross section 8-8 presented in Figure A2.5-31 (location identified in Figure A2.5-2) is located at the highest terrain along the north edge. The configuration of the heat sink's cross section following postulated sliding is shown on Section 8-8 where the cross-hatched area indicates material that has slid into the heat sink. The toe of the slide material lies 280 feet from the toe of the baffle dike. At the east and west end of the UHS, no change in the surface area is expected due to the stable till slopes and compacted dike respectively.

d. Loss of ultimate heat sink water through the earth retaining fill cannot occur because:

1. The fill is compacted cohesive soil not susceptible to liquefaction.

2. The fill was founded on Illinoian glacial till and it abuts Illinoian glacial till which forms the valley walls.

3. The fill is protected with soil cement to withstand extreme event flow velocities.

4. The fill is designed to be stable under seismic loading.

It is, therefore, concluded that the ultimate heat sink will provide sufficient water volume and cooling capability for the station for 30 days with no water makeup.

A2.5.6 References

1. I. M. Idriss and D. M. Hendron, "Potential for Alluvium in North Fork of Salt Creek to Flow into Heat Sink and Prevent Cooling Water Flow, Task I," Technical Report, Woodward-Clyde Consultants, April 30, 1975.

2. Transcripts of the ACRS 180th General Meeting, Washington, D.C., pp. 192-230, April 4, 1975.

3. J. P. Kepton, 1975 Illinois State Geological Survey, telephone communication, R. W. Wagner, April 21, 1975.

ATTACHMENT B2.5

SAND LENSES UNDER CATEGORY I STRUCTURES

ATTACHMENT B2.5 - SAND LENSES UNDER CATEGORY I STRUCTURES

B2.5 SAND LENSES UNDER CATEGORY I STRUCTURES

B2.5.1 Summary

Postulating the liquefaction of sand lenses under Category I structures the following information was requested at the PSAR stage and is presented in this attachment.

a. Evaluate the total and differential settlement caused by liquefaction of the sand lenses under earthquake loading. Consider both the regional SSE and the long duration event located in the Wabash Valley. Discuss the effect of this settlement on safety-related Category I foundations. State or reference the structural design criteria for settlement and differential settlement, and show that these criteria will not be exceeded.

b. Describe the remedial treatment proposed for assuring that liquefaction of these sand lenses under earthquake loading will be presented.

Soil borings, laboratory test data, relative densities based on standard penetration test values, grain size analyses, material type, in situ dry densities, and confining pressures have led to the conclusion that the sand lenses do not have characteristics of sand deposits capable for liquefaction under the postulated earthquake loadings; therefore, no remedial measures are needed and no settlements anticipated.

B2.5.2 Subsurface Conditions

The subsurface conditions in the area of the station mat foundations have been investigated with a series of borings; the boring logs are presented in Figures 2.5-19 through 2.5-73. Those borings which penetrate the area within the outline of the station's mat foundations are located in Figure B2.5-1.

The soil under the station site and above approximately elevation 683 feet was removed and the excavation filled with Type B granular material recompacted to 85% relative density as determined by ASTM D-2049.

The Illinoian glacial till beneath the station site was located between elevations 575 and 686 feet, and the boring logs showed scattered sand lenses within the upper 30 feet of the Illinoian till. The strata in which the sand lenses lie is a glacial outwash area formed during the Illinoian glacial retreat. In outwash zones there is a high probability for the formation of scattered sand and gravel lenses and layers during fluctuation of the river flowing from the glacier.

The geological process involved in the formation of the top 30 feet of the Illinoian till accounts for the apparent random scattering of granular material within the upper strata of the till. Figure B2.5-2 presents a plot of all granular material that has been penetrated by borings located within the outline of the station's mat foundation. Although the granular material cannot be definitely identified as lying in specific pockets and lenses, for purposes of discussion the granular material has been identified as lying in six distinct lenses as discussed herein.

B2.5.2.1 Additional Geotechnical Investigation

In order to verify the assumptions made in determining the characteristics of sandy soil forming these lenses, additional geotechnical investigation consisting of seven soil borings was performed. The seven borings (P-33A, P-50 through P-55) are located in Figure B2.5-1 and the subsurface conditions are shown on the boring logs in Figures B2.5-3 through B2.5-9.

The program consisted of four primary borings (P-33A, P-50, P-52, and P-54) in which continuous sampling and standard penetration test values were taken in the sand lenses by means of a split spoon. The secondary borings located by P-51, P-53, and P-55, were primarily intended to retrieve sand samples for laboratory testing. The samples in the secondary holes were obtained by either pitcher sampler or Dames & Moore 'U' sampler. Modified Osterberg sampler was also used to try to obtain undisturbed samples, but due to dense sand deposits no samples could be recovered.

The laboratory testing program consisted of particle size analysis including hydrometer, Atterberg limit, and dry densities on relatively undisturbed samples. The results of the tests are shown on boring logs and Figures B2.5-10 through B2.5-15.

B2.5.3 Soil Characteristics Influencing Liquefaction

It is an established fact that liquefaction potential of soil deposits due to earthquake motion depends on characteristics of the soil, the initial stresses acting on the soil, and the characteristics of the earthquake involved (Reference 1). Significant factors include:

a. The relative density

Relative density is the most important physical characteristic that determines the liquefaction potential of a soil. The higher the relative density, the less susceptible the soil is to liquefaction.

b. The soil type

Fine sands and fine to medium sands tend to liquefy more easily than do coarse sands, gravelly soils, fine silts or clays. There is some evidence to show the poorly graded materials are more susceptible to liquefaction than well graded materials.

c. The initial confining pressure

The liquefaction potential of a soil is reduced by an increase in confining pressure. State-of-the-art evaluation of soil characteristics for seismic response analyses (Reference 2) states, "From field observations it has generally been concluded by a number of investigators that even in a saturated sand deposit below a depth of 50 to 60 feet, sands are not likely to liquefy. These depths are in general agreement with Kishida (Reference 3) who states that a saturated sandy soil is not liquefiable if the value of the effective overburden pressure exceeds 2 kg/cm^2 (kg/cm^2 $\underset{\sim}{}$ 60 ft of soil below water table $\underset{\sim}{}$ 4.1 kips/sq ft)."

Characteristics of earthquakes for this site are defined in Subsection 2.5.2.6.

B2.5.4 Liquefaction Potential of Sand Lenses

The sand lenses under the main plant foundation are characterized by appreciable fines (passing U.S. sieve No. 200) ranging from 20% to as much as 55%. The high blow counts in the lenses are substantiated by high values of dry densities obtained from the extracted samples on relatively undisturbed samples in the laboratory.

Shown in Figure B2.5-2 are the sand lenses under the main plant foundations. The sand lenses have been plotted to show the elevations at which these have been found and the corresponding boring numbers. A study of the sandy soil under the plant was performed which utilized the following parameters in delineating sand lenses:

a. soil description,

b. soil classification,

c. penetration values, and

d. elevation at which the lenses exist.

By studying these four parameters, the sand beneath the station can be classified into six distinct lenses. The characteristics of these lenses have been tabulated in Table B2.5-1.

Sand Lens No. 1 This sand lens is located by Boring P-32 between elevations 675 feet and 677.5 feet. The lens has a Dames & Moore sampler penetration value of 10. The material is described as "Brown and gray, medium to coarse sand with trace of fine sand and silt," and is classified as SP-SW material. This lens was investigated after the foundation excavation and was completely removed.

Sand Lens No. 2 A lens of SM material described as "Gray silty fine to coarse sand with a trace of fine gravel" is evident in Boring P-34. The extent of this layer is from elevation 677 feet to 679.5 feet. The standard penetration test N value exceeds 200, indicating extremely dense consistency.

As shown in Table B2.5-1, this sand has a relative density exceeding 95% and therefore is not susceptible to liquefaction.

Sand Lens No. 3 This lens underlies most of the main plant site and appears in Borings P-10, P-14, P-30, P-33, P-34, P-35, P-36, P-37, P-41, P-33A, and P-52. This lens extends from elevation 652 feet to 665.5 feet. The soil description generalized from the 15 samples is "Gray fine to coarse sand with some silt and fine gravel" and can be classified as SM-SP-SW material.

A major portion of sand under the main plant structures is included in this lens, and has a mean elevation of 658 feet. In order to determine the representative relative density of the lens, a statistical analysis was performed on the various penetration values and the result of this study has been summarized in Table B2.5-1.

Statistical Property Evaluation of Sand Lens

To accurately represent the soil properties in the large sand lens, shown in Figure B2.5-2, a statistical analysis was performed. This type of analysis allows for variations in the testing procedure and will yield a probabilistic range of values.

The first step in this procedure is to reduce the field test data. Nineteen standard penetrations and Dames & Moore samples were taken in a sand lens, which appears, by soil description and elevation similarity, to be the same lens. The corrected standard penetration N-Values were used to statistically compute a mean value to be used in relative density calculations.

The analysis was performed assuming a normal distribution for the N-Values within the layer. There is a 97.5% probability that the five mean values will be greater than 68 blows per foot. This analysis was performed with the methods shown in Reference 4.

Utilizing the unit weights for tills and the distances indicated on the boring logs, the vertical effective overburden pressure was found for each of the 19 samples.

A statistical analysis of these two values was performed to achieve a confident range of values to use in the relative density calculations. A range of the arithmetic mean ± one standard deviation of the samples was used and this yielded a range of 5.68 to 6.40 ksf.

With the N value and vertical effective overburden pressure, the Gibbs & Holtz relationship (Reference 5) for relative density was used. A range of values, based on the range of effective overburden pressures, was achieved. This range was from 87% to 92% relative density, and an average relative density of 90% was assigned to Lens 3. Sands with a relative density of 90% are unlikely to liquefy under the given confining pressures and anticipated loading (Reference 3).

Sand Lens No. 4 Boring P-10 indicated a lens of "Gray fine to coarse sand with trace silt" (SW) from elevation 672.5 feet to 669 feet. The split spoon sample showed a standard penetration test value of 64 which indicates a material of very dense consistency.

As shown in Table B2.5-1, this sand lens has a relative density exceeding 90%, and therefore is not susceptible to liquefaction.

Sand Lens No. 5 This sand lens is located north of the plant site and was originally shown in Borings P-33 and P-41. In order to better define this lens, three primary borings (P-33A, P-50, and P-52) were drilled at the locations shown in Figure B2.5-1, and continuous sampling of the sand lens was done with 'N' values taken at 1.5 feet intervals. As shown by the boring logs, P-33A and P-50, the sand lens does not extend under the station mat at the northeast section of the plant. Boring P-52 shows very dense sand from elevation 662.5 feet to 675.0 feet with an average standard penetration value of 71 (average of seven values).

The sand in this lens is mostly medium to coarse sand with little gravel and silt; it is identified as well graded material (SW-SM-SP). Based on the standard penetration values of 71 and using the criteria proposed by Gibbs & Holtz, the relative density of the lens is approximately 95%. The characteristics of liquefaction in this sand lens deposit are absent; hence, it is not susceptible to liquefaction.

Sand Lens No. 6 The grain size analysis shown in Figure B2.5-16 indicates fines up to 55% in the upper part of the lens and, as indicated by Reference 6, it is not susceptible to liquefaction. The low blow counts of 23 and 13 are inconsistent with the dry density of 133 pcf obtained from the laboratory testing of the sample and, therefore, the criteria of relative density is not applicable.

This lens, previously shown only in Boring P-38 was examined by the primary Boring P-54 taken in the vicinity as shown in Figure B2.5-1. Based on the sieve analysis data presented in Figure B2.5-16, the material can be classified as ML-SM and has a percent passing the U.S. sieve No. 200 up to 45% in the lower portion of the lens. The soil Boring P-54 taken in the vicinity of this sand lens has standard penetration values N of 81 and 50. The average standard penetration N value of this sand lens is 50 and has a relative density of 85%. Based on this information, we conclude that this sand lens is not susceptible to liquefaction.

B2.5.5 Conclusions

Sand lenses under the plant site have been examined for:

a. relative density,

b. soil type, and

c. initial confining pressure.

From the numerous studies conducted on sands both in the laboratory and field, these three are the principal soil characteristics effecting liquefaction of sand deposits under earthquake loading. The properties of sand under the foundation mat have been examined for these characteristics and, in all cases, it has been found that the lenses will not liquefy under the earthquake loading.

The consistency of the lenses was based on standard penetration test values; all the lenses (except Lens 1) have a relative density greater than 85%. The dense nature of the sands is borne out by a limited number of in situ dry density values of sands (P-6, P-13, P-15, P-20, P-26, P-43, H-16) where dry densities are found to be more than 122 pcf which are greater than 95% relative density of Sangamonian sands in the area. During the excavation and subgrade testing, Lens 1 was completely removed.

The material of the sand lenses is usually medium to coarse sands, with some fine gravel and silts, and is usually well graded material, making it resistant to liquefaction process.

The sand lenses are at a depth exceeding 60 feet and the effective overburden pressure is more than 5.5 kips per square foot with the addition of the plant foundation loads, the effective pressure on these lens could exceed by more than two times the pressure (4.1 kips per square foot), which according to Kishida (Reference 3) will prevent saturated sandy soils from liquefying.

It is, therefore, concluded that sand lenses will not liquefy under the earthquake loading and consequently, no settlement due to liquefaction is anticipated.

B2.5.6 References

1. H. B. Seed and I. M. Idriss, "A Simplified Procedure for Evaluating Soil Liquefaction Potential," Report No. EERC 70-9, University of California, Berkeley, California, November 1970.

2. Shannon & Wilson, Inc. and Agbanian-Jacobsen Associates, "Soil Behavior Under Earthquake Loading Conditions: State of the Art Evaluation of Soil Characteristics for Seismic Response," prepared for the U.S. Atomic Energy Commission, January 1972.

3. H. Kishida, "Characteristics of Liquefied Sands During Mino-Owari, Tohnankai, and Fukui Earthquakes," Soils and Foundations (Japan), Vol. 9, No. 1, pp. 75-92, March 1969.

4. Benjamin and Cornell, "Probability, Statistics and Decision for Civil Engineers."

5. H. J. Gibbs and W. G. Holtz, "Research on Determining the Density of Sand by Spoon Penetration Testing," Proceeding of 4th International Conference on Soil Mechanics and Foundation Engineering, London, Vol. I, pp. 35-39, 1957.

6. E. D'Appolonia, "Dynamic Loadings," Journal of Soil Mechanics and Foundations Division, ASCE, No. SM1, Volume 96, p. 61, January 1970.

TABLE B2.5-1
SAND LENS SUMMARY

LENS NUMBER	ELEVATION (ft-in.) FROM	ELEVATION (ft-in.) TO	UNIFIED CLASSIFICATION	MATERIAL SOIL DESCRIPTION	MATERIAL BORINGS CONSIDERED	SPT REPRESENTATIVE BLOWCOUNT	RELATIVE DENSITY**	% FINER #200 SIEVE	REMARKS
1	675	677.5	SP-SW	Brown and gray med-coarse sand with trace fine sand and silt	P-32	9 (10 = D&M)	104.4***	N.A.	1) Tested and approved in field 2) Unit WT = 133.5 pcf dry density 3) Confining pressure
2	677	679.5	SM	Gray silty fine to coarse sand with trace fine gravel	P-34	200 / 6 in.	95+	N.A.	1) Gradation 2) High R.D. 3) Confining pressure
3	652	665.5	SM-SP-SW	Gray fine to coarse sand with some silt and fine gravel	P-10, P-14, P-30 P-33, P-34, P-35 P-36, P-37, P-41 P-52, P-33A, P-15	68*	90+*	38-52% (From P-14, P-15 Grain Size Analysis)	Based on statistical analysis Will not liquefy 1) Due to high R.D. 2) Due to high % fines 3) Due to gradation 4) Confining pressure
4	669	672.5	SW	Gray fine to coarse sand with trace silt	P-10	64	90	N.A.	1) High R.D. 2) Gradation 3) Confining pressure
5	663.5	760.5	SW-SM-SP	Gray silty fine sand with some fine to coarse and fine gravel	P-33, P-41, P-52	70 (AVG of 7)	95		1) High R.D. 2) Gradation 3) Confining pressure
6	661	667	SM	Gray silty fine sand	P-38, P-54	50	85	45 to 55	See Figure 2.5-17.29 1) Will not liquefy due to high % fines 2) Confining pressure 3) R.D.

* Based on statistical analysis with 97.5% probability.
** Based on Gibbs & Holtz criteria.
*** Based on inplace field testing.

ATTACHMENT C2.5

GEOLOGIC MAPPING PROGRAM FOR THE POWER BLOCK, ULTIMATE HEAT SINK, SCREEN HOUSE, AND SHUTDOWN SERVICE WATER SYSTEM EXCAVATIONS

ATTACHMENT C2.5 - GEOLOGIC MAPPING PROGRAM FOR THE POWER BLOCK, ULTIMATE HEAT SINK, SCREEN HOUSE, AND SHUTDOWN SERVICE WATER SYSTEM EXCAVATIONS

C2.5.1 Introduction

This attachment presents the results of a geologic mapping program conducted during the periods March 15-19, 1976, at the power block excavation; July 20-23, 1976, at the ultimate heat sink dam; May 24-26, 1977, at the baffle dike abutment; and June 16-23, 1977, at the screen house and outlet structure excavations. The purposes of the program were to verify the site stratigraphy as determined by the boring program and as described in the Preliminary Safety Analysis Report (PSAR), to gather data for preparation of the Final Safety Analysis Report (FSAR), and to confirm that the floor of the excavations extended into unaltered, Illinoian till as stated in the PSAR.

Site stratigraphic units (all Pleistocene in age) discussed in this report are the Peyton Colluvium; the Cahokia Alluvium Henry Formation; the Richland Loess; the Wedron Formation; the Robein Silt; the weathered Glasford Formation; and the unaltered Glasford Formation. Descriptions and ages of each of the stratigraphic units are presented in Figure C2.5-1. The stratigraphic nomenclature has been refined from that presented in the PSAR: however, it does not indicate any difference between the lithologic units encountered in the borings and those mapped in the excavations. A comparison of terminology used in this Attachment and the PSAR is presented in Figure C2.5-2.

Figure C2.5-3 shows locations of all major excavations referred to in the text. Figure C2.5-4 presents a plan view of the power block excavation showing the locations of main structures within the power block and the locations of the four excavation walls. Figure C2.5-5 shows the location of the screen house and the four excavation walls. Figure C2.5-6 gives a plan view of the ultimate heat sink area and shows the locations of the submerged dam, baffle dike, screen house, and shutdown service water system (SSWS) outlet structure.

C2.5.2 Conclusions

The geologic mapping program confirmed that:

a. The lithologic units exposed in the excavations are the same as those encountered in the borings and presented in the PSAR. One exception was the discovery of a black organic silt lens in the baffle dike abutment and submerged dam abutment excavations.

b. The stratigraphic units are continuous across the excavations.

c. Sand deposits within the tills, which were interpreted from the boring data as being discontinuous pockets or lenses, were exposed in the excavations and were confirmed to be discontinuous pockets. Two exceptions were noted: a nearly continuous, 2 to 3 feet thick, layer of brown, fine sand occurring within the Wedron Formation at approximately elevation 725 feet MSL in the power block and screen house excavations (Figures C2.5-7 through C2.6-12), and a continuous layer of fine to coarse sand at approximately elevation 658 feet MSL on the south abutment of the submerged dam (Figure C2.5-13).

d. The floor of the excavations extended into unaltered Illinoian till of the Glasford Formation as stated in the PSAR.

e. No geologic features showing vertical offset of stratigraphic units were noted.

The foundation design assumptions presented in the PSAR are valid, and no changes based on soil conditions encountered in the excavations were required in the design of the power block and ultimate heat sink structures, with the exception of minor design changes at the abutments of the submerged dam and baffle dike.

C2.5.3 Scope

The geologic mapping program consisted of establishing detailed descriptions of the site stratigraphic units; identifying, staking, and surveying contacts between these units at selected points within the excavations; and preparing geologic sections and photo mosaics (power block excavation) to document the mapping program.

T. C. Buschbach, H. B. Willman, D. L. Gross, L. R. Follmer, and C. S. Hunt of the Illinois State Geological Survey (ISGS) visited the site on Thursday, March 18, 1976, to inspect the stratigraphic units and contacts exposed in the power block excavation and to verify the results of the geologic mapping program. The ISGS agreed with Sargent & Lundy's identifications of the stratigraphic units and contacts exposed in the power block excavation (T. C. Buschbach, written communication to L. J. Koch, Illinois Power Company, April 9, 1976).

J. W. Skrove, U.S. Nuclear Regulatory Commission, visited the site on Tuesday, April 20, 1976, to inspect the stratigraphic units and contacts exposed in the power block excavation (J. W. Skrove, written communication to W. P. Gammill, DSE, April 27, 1976).

C2.5.3.1 Procedure

The geologic mapping consisted mainly of a visual inspection of each of the excavation walls. Hand tools were used to trench into relatively undisturbed soil. In areas where localized sloughing had occurred near the floor of the power block excavation, a backhoe was used to dig trenches to expose the undisturbed soil. Descriptions of the stratigraphic units were prepared and stratigraphic contacts were staked and labelled. Geologic contacts separating the Wedron Formation, the Robein Silt, the weathered Glasford Formation, and the unaltered Glasford Formation were identified on the walls of the excavations.

After identifying the stratigraphic units and contacts, the Sargent & Lundy mapping team located major sand deposits on the excavation walls. This was accomplished by visual observation of color and erosional patterns, investigation of areas of obvious groundwater seepage, and hand trenching on the excavation walls. After all staking had been completed, surveyors established the locations of the staked control points to the nearest 0.01 foot using plant grid coordinates and the elevations of the staked control points to the nearest 0.2 foot using mean sea level datum.

C2.5.4 General Site Stratigraphy

Stratigraphic units present at the power block, ultimate heat sink, screen house, and shutdown service water system (SSWS) excavations were the Peyton Colluvium; the Cahokia Alluvium Henry Formation; the Richland Loess; the Wedron Formation; the Robein Silt; the weathered Glasford Formation; and the unaltered Glasford Formation. Not a11 formations were present in all places. See Section C.2.5.5 and Figures C2.5-1 and C2.5-2 for ages of the formations and lithologic details.

The Peyton Colluvium consists of brown clayey silt with minor amounts of gravel along the base of the slopes of the North Fork of Salt Creek. The colluvium was entirely removed from the submerged dam abutments, baffle dike abutment, screen house, and ECCS outlet structure excavations.

The Cahokia Alluvium - Henry Formation consists of alluvial and outwash deposits and is confined to the valleys of Salt Creek and North Fork of Salt Creek. The alluvium is primarily poorly sorted silt, clay, and silty sand with lenses of sand and gravel. Underlying the alluvium is glacial outwash of the Henry Formation which consists of yellow-brown fine to coarse sand and gravel with pockets of gray and brown silt, sandy silt, and silty clay. A lag gravel is often present at the base of the Henry Formation. The contact between the Cahokia Alluvium and Henry Formation was not mapped because nearly all of the Cahokia Alluvium was removed from excavations where originally present and little of significance could be gained by mapping what remained. The contact between the Cahokia Alluvium - Henry Formation and Glasford Formation (weathered or unaltered) was identified by an abrupt change in color and decrease in grain size.

The Richland Loess is almost entirely confined to the upland areas and consists of brown clayey silt, with trace fine sand. The loess had been removed or highly disturbed over most of the perimeter of the power block and screen house excavations prior to mapping. The contact with the underlying Wedron Formation was not identified.

The Wedron Formation is almost entirely confined to the upland areas and consists of Wisconsinan till, which is brown to gray clayey silt to silty clay with some fine sand and trace gravel. Discontinuous lenses of brown to gray, fine to coarse sand occur within the till. The contact between the Wedron Formation and the underlying Robein Silt was defined by an abrupt change in color, clay content, and presence of organic material.

The Wedron Formation is a stable material on construction slopes of 1.5 horizontal to 1 vertical and, in general, makes excellent backfill. Locally, the glacial till includes sand pockets and lenses generally with water seeps. The sands are easily eroded by water flowing down the slope.

The Robein Silt consists of water-deposited loess characteristically with dark brown organic silt and traces of clay and fine sand. Locally, some peat is also found. The Robein Silt is confined to the upland areas.

The Robein Silt has a tendency to break into vertical slopes, especially under freeze-thaw conditions. Included organic material and clay, acting as a binder, reduces erosion by running water to a minimum.

The weathered Glasford Formation consists of weathered Illinoian till that was weathered during Sangamonian and later times. It is characterized by gray silt, with trace fine sand grading to gray-green silty clay or clayey silt, with some fine to coarse sand and trace fine to coarse gravel. The weathered Glasford Formation is absent in the valleys. The weathered till is slightly to highly calcareous. The contact within the Glasford Formation, between the weathered and unaltered till, is gradational. Based on the site visit by the ISGS, this contact was defined where a very strong reaction to dilute muriatic acid (HCl) was achieved, where no noticeable color change occurred with increased depth, and where a normal gravel fraction was present in the till.

The weathered Glasford Formation is stable on slopes of 1.5 horizontal to 1 vertical. A few seeps, associated with discontinuous sand lenses, were noted.

The unaltered Glasford Formation consists of unaltered Illinoian till and is characterized by a dark gray clayey silt with some interspersed fine to coarse sand and fine to coarse gravel. The till is highly calcareous and contains discontinuous lenses of gray, fine to coarse sand. Unweathered Glasford Formation is present everywhere on the site. Unweathered Glasford Formation is a very hard material stable on slopes of 1.5 horizontal to 1 vertical. Some sand pockets were noted. In the ultimate heat sink excavation, a silt layer similar to the Robein Silt was exposed at about elevation 660, some 20 feet below the top of the unweathered Glasford Formation.

C2.5.5 Findings

C2.5.5.1 Power Block Excavation

Stratigraphic units presented at the power block excavation, in descending order, were: the Richland Loess, highly disturbed due to construction operations; the Wedron Formation; the Robein Silt; the weathered Glasford Formation; and the unaltered Glasford Formation.

Figures C2.5-7 through C2.5-10 show that, with the exception of the southwest corner of the excavation, the contact between the Wedron Formation and the underlying Robein Silt was generally continuous at approximately elevation 698 feet MSL. In the southwest corner (Figures C2.5-9 and C2.5-10), the Robein Silt and the underlying, weathered Glasford Formation had been eroded almost completely away by stream action during the early part of the Wisconsinan Age. The contact between the weathered and unaltered portion of the Glasford Formation was at a fairly constant elevation of approximately 685 feet MSL.

Sand deposits were exposed in the walls of the excavation within the Wedron Formation and near the contact between the weathered and the unaltered portions of the Glasford Formation. The sand deposits within the Wedron Formation consisted of a nearly continuous seam of brown, fine sand generally 2 to 3 feet thick that spanned all four walls of the excavation at approximately elevation 725 feet MSL and thin, discontinuous seams and localized pockets of gray, fine to coarse sand within the till, extending several feet vertically and horizontally. Sand deposits were difficult to identify on the south and east excavation walls due to sloughing and continuing excavation by dragline. The geologic sections in Figures C2.5-7 through C2.5-10 show both the sand deposits identified in the field and those established from the interpretation of photographs.

Sand deposits were exposed near the contact between the weathered and the unaltered portion of the Glasford Formation on the east, south, and west walls of the excavation. These sand deposits were variable in texture and discontinuous in extent and carried varying amounts of groundwater into the excavation. Along the east wall (Figure C2.5-8), reference points marked with "X's" within the unaltered Glasford Formation at stations 4+71 North and 7+49 North mark the locations of small sand lenses from which groundwater was seeping at a perceivable rate into the excavation.

The floor of the excavation was located entirely within the unaltered Illinoian till of the Glasford Formation in all areas inspected by the mapping team. The elevation of the floor varied across the excavation because of over-excavation to remove sand pockets within the unaltered Glasford Formation and to provide drainage to a common sump. The entire floor of the excavation had not been cleaned at the time of field mapping to provide a fresh exposure of the unaltered Illinoian till of the Glasford Formation. A systematic testing program and visual inspection by onsite personnel were performed to verify the presence of the unaltered Glasford Formation on the floor of the excavation prior to fill placement.

C2.5.5.2 Screen House Excavation

Stratigraphic units exposed on the walls of the screen house excavation included the Cahokia Alluvium - Henry Formation; the Wedron Formation; the Robein Silt; and the Glasford Formation, consisting of a weathered and unaltered zone (see Figures C2.5-11, C2.5-12, C2.5-14, and C2.5-15). Richland Loess (not mapped) was exposed locally, but was highly disturbed by construction activities.

The Cahokia Alluvium - Henry Formation is exposed on the south and west walls (Figures C2.5-14 and C2.5-15) of the screen house excavation, where the excavation extends into the flood plain of the North Fork of Salt Creek. Mapping was performed after the ultimate heat sink was excavated to elevation 668.5 feet MSL; consequently, the upper part of the Cahokia Alluvium - Henry Formation was not exposed at the time of mapping. The alluvial and glacial outwash deposits occur from the surface at elevation 680 feet MSL to approximately elevation 660 feet MSL, where they are underlain by the unaltered Glasford Formation.

On the north and east walls of the screen house excavation (Figures C2.5-11 and C2.5-12), the youngest stratigraphic unit was the Wedron Formation. In the Wedron Formation, a nearly continuous seam of brown, fine sand, generally 1 to 2 feet thick was exposed on the east wall of the excavation at approximately 725 feet MSL. Thin, discontinuous seams and lenses of gray, fine to coarse sand were noted within the Wedron Formation. The contact between the Wedron Formation and the underlying Robein Silt was generally continuous at approximately elevation 686 feet MSL.

Illinoian till of the Glasford Formation, consisting of a weathered and unaltered zone, underlies the Robein Silt at approximately elevation 684 feet MSL. The contact between the weathered and unaltered zones was gradational at approximately elevation 680 feet MSL. Discontinuous sand units were exposed near the contact between the weathered and unaltered Glasford Formation on the east and north walls of the excavation. Groundwater flow from these sand units was minor and variable. The floor of the excavation at elevation 653 feet MSL was located entirely within the unaltered till. Some overexcavation was necessary to remove discontinuous sand lenses and to provide drainage to a common sump.

C2.5.5.3 Ultimate Heat Sink

C2.5.5.3.1 Submerged Dam and Baffle Dike Excavations

Excavations for the submerged dam and baffle dike in the heat sink bottom revealed the Cahokia Alluvium - Henry Formation overlying hard glacial till of the unaltered Glasford Formation. Mapping was performed after the heat sink bottom was excavated to elevation 668.5 feet MSL; consequently, the upper part of the Cahokia Alluvium - Henry Formation was not present at the time of mapping. Geologic profiles (Figures C2.5-16 and C2.5-17) indicate the contact between the Cahokia Alluvium - Henry Formation and the underlying unaltered Glasford Formation is continuous at approximately elevation 658 feet MSL for the submerged dam and approximately elevation 660 feet MSL for the baffle dike.

C2.5.5.3.2 Submerged Dam Abutments

Stratigraphic units exposed in the abutments of the submerged dam in descending order are: the Richland Loess, mappable only on the south abutment; the Wedron Formation; the Robein Silt; and the Glasford Formation, consisting of weathered and unaltered zones. Figures C2.5-18 and C2.5-19 present a plan view of the north abutment and south abutment, respectively.

The contact between the Richland Loess and the Wedron Formation varies with the topography along the south abutment (Figure C2.5-13). The contact between the Wedron Formation and the underlying Robein Silt is continuous at approximately elevation 696 feet MSL for the south abutment and elevation 702 feet MSL for the north abutment (Figure C2.5-20). The contact between the Robein Silt and the underlying weathered portion of the Glasford Formation is generally continuous at approximately elevation 694 feet MSL for the south abutment and elevation 699 feet MSL for the north abutment. The contact between the weathered and unaltered portions of the Glasford Formation varies due to the proximity to the natural ground surface. Geologic mapping verified unaltered Glasford Formation below elevation 676 feet MSL.

Sand deposits are exposed in the abutments within the Wedron and Glasford Formations. The sand deposits within the Wedron Formation consist of local pockets of brown, fine to coarse sand. The sand deposits within the Glasford Formation are generally discontinuous seams and localized pockets of gray, fine to coarse sand. Between elevation 660 and 664 feet MSL, silt and sand seams, interbedded with a 1 to 2 foot thick layer of organic silt, were noted within the unaltered Glasford Formation. Seepage of groundwater was noted within the sand directly above the unaltered Glasford Formation.

On the north abutment the sand seams were not present and organic silt at approximately elevation 663 MSL was overlain by unaltered Glasford Formation and was underlain by a thin strata of weathered Glasford Formation, which grades into unaltered Glasford Formation.

C2.5.5.3.3 <u>Baffle Dike Abutment</u>

Stratigraphic units exposed in the baffle dike abutment in descending order were: the Richland Loess, which was highly disturbed due to construction operations; the Wedron Formation; the Robein Silt; and the Glasford Formation, consisting of weathered and unaltered zones.

Figure C2.5-21 shows a geologic section of the baffle dike abutment. The contact between the Wedron Formation and the underlying Robein Silt is continuous at approximately elevation 687 feet MSL. The Robein Silt is underlain by the weathered Glasford Formation at an elevation of about 686 feet MSL. The contact between the weathered and unaltered zones of the Glasford Formation was mapped at approximately elevation 676 feet MSL. The unaltered Glasford Formation between elevation 676 feet MSL and 668 feet MSL was soft to medium stiff, and the till below elevation 668 feet MSL was hard. A large sand lens in the Glasford Formation pinches out on the north side of the baffle dike abutment but continues on the south side of the dike. Groundwater emerged from the lower part of this sand deposit. Two thin discontinuous lenses of black organic silt were noted at approximately elevations 662 feet MSL (upper) and 651 feet MSL (lower) near the large sand lens at the north and south ends of the excavation.

C2.5.5.4 <u>SSWS Outlet Structure</u>

Stratigraphic units exposed to the SSWS outlet structure excavation include: Peyton Colluvium, which is exposed at the base of the bluffs of the North Fork of Salt Creek; the Cahokia Alluvium - Henry Formation; and the Glasford Formation. Figure C2.5-22 is a geologic section of the east wall of the excavation; the other walls of the excavation revealed Cahokia Alluvium - Henry Formation deposits underlain by unaltered Glasford Formation.

Peyton Colluvium was mapped along a sloping contact with the underlying Glasford Formation between elevation 672 and 679 feet MSL. Within the Glasford Formation at the SSWS outlet structure excavation were two organic silt units. The upper silt unit was exposed near the contact with the Peyton Colluvium from elevation 670 feet MSL (north) to elevation 678 feet MSL (south). The lower silt unit occurred at approximately elevation 662 feet MSL, near the floor of the excavation.

C2.5.5.5 SSWS Piping Subgrade

The SSWS piping excavations extended from the screen house and SSWS outlet structure to the south side of the power block excavation (Figure C2.5-23).

The subgrade for the SSWS pipes consists mainly of Wisconsinan till of the Wedron Formation. Near the screen house and SSWS outlet structure, the Robein Silt was exposed in the piping excavation at approximately elevation 688 feet MSL. The silt was partially removed by overexcavation and the excavation was raised to the proper level with suitable backfill material. Several thin, discontinuous sand and silt lenses and small randomly distributed sand and silt pockets were noted in the Wedron Formation.

ATTACHMENT D2.5

UNPUBLISHED NOTES

PITTSFIELD-HADLEY ANTICLINE

The Pittsfield-Hadley Anticline is a prominent structure that crosses the Mississippi River Arch in a northwest-southeast direction. It is located in Lewis County, Missouri, and Pike County, Illinois. It plunges southeastward and it appears to lose its identity in Greene County, Illinois. Pennsylvanian strata on the flanks at the anticline dip somewhat less than underlying Mississippian strata. This indicates that some uplift occurred during post-Mississippian and pre-Pennsylvanian time, but the major uplift took place after Pennsylvanian time. Total uplift along the anticline exceeds 300 feet in some places. Folds with similar directional trends, but with uplift of only slightly more than 100 feet occur at Fishhook in Pike County and at Media in Henderson County, both in Illinois. In view of the similarities of their orientation and their stratigraphy, these minor structural highs are assumed to have formed at the same time as did the Pittsfield-Hadley anticline.

T. C. Buschbach 5/73

Ref. F. Krey, Structural Reconnaissance of the Mississippi Valley Area from Old Monroe. Missouri to Nauvoo, Illinois: Illinois Geol. Survey Bull. 45., 1924.

MISSISSIPPI RIVER ARCH

The Mississippi River Arch is a broad, corrugated fold which extends generally north-south through the bulge of western Illinois. To the north it blends with the Wisconsin uplands and to the south it intercepts the Lincoln Anticline. The arch separates the Illinois Basin from the Forest City Basin. Dating of movements along the arch is difficult because erosion has removed the Pennsylvanian strata. However, it appears that the Mississippi River Arch existed early in Pennsylvanian time and was probably subjected to additional deformation at the end of Paleozoic time. The arch is cut by numerous cross folds which trend northwest-southeast and plunge gently southeastward in the Illinois Basin.

T. C. Buschbach 5/73

Ref. J. V. Howell, The Mississippi River Arch: Kans. Geol. Soc. Guidebook, 9th Annual Field Conf., pp. 386-389, 1935.

SALEM-LOUDEN ANTICLINAL BELT

The Salem-Louden Anticlinal Belt is a prominent structural high in the Illinois Basin. The anticlinal belt trends northnortheast and is most prominent in Central Marion and eastern Fayette Counties. Closure of 100 feet or slightly more is common on the anticline, and the structure has proved to be important in determining the position of oil fields. Individual units within the Pennsylvanian System thin over the top of the anticlinal belt, indicating that the structure was uplifted during Pennsylvanian time. Uplift also continued after Pennsylvanian deposition ended.

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Ref. E. P. DuBois, Geology and Coal Resources of a Part of the Pennsylvanian System in Shelby, Moultrie, and Portions of Effingham and Fayette Counties: Illinois Geol. Survey Rept. Inv. 156, 1951.

SANGAMON ARCH

The Sangamon Arch was formed by uplift in central and western Illinois during Devonian and early Mississippian time. The arch extends from the Mississippi River Arch eastward to Macon and DeWitt Counties in central Illinois. Its limits are reasonably well defined by the zero isopach of the Cedar Valley (Middle Devonian) Limestone. Although several hundreds of feet of Devonian and Silurian strata normally present in surrounding areas were either not deposited over, or eroded from the arch, later movements have masked the arch so that it does not show on structure maps of the area. It is a relict structure that is interpreted by the stratigraphy of the region.

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Ref. L. L. Whiting, and D. L. Stevenson, The Sangamon Arch: Illinois Geol. Survey Circ. 383, 20 pp., 1965.

DUPO-WATERLOO ANTICLINE

The Dupo-Waterloo Anticline strikes north-northwest from Monroe County, Illinois, through St. Louis, Missouri, and appears to terminate against the Cap au Gres faulted flexure about 12 miles north of St. Louis. Outcrops in the Dupo area show that the east flank dips 2 to 3 degrees, whereas the west flank dips up to 30 or more degrees. The Dupo-Waterloo anticline was probably intermittently active from Silurian time to post-Pennsylvanian time. Major movement appears to have occurred near late Mississippian, pre-Pennsylvanian time, with renewed uplift in post-Pennsylvanian, pre-Pleistocene time. Total strucutral relief is at least 500 feet near Waterloo. (Precambrian high?)

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Ref. A. H. Bell, The Dupo Oil-Field: Illinois Geol. Survey, Illinois Petroleum 17, 1929.

SANDWICH FAULT ZONE

The Sandwich Fault Zone strikes across northern Illinois in a west-northwest direction from western Will County to Ogle County. It forms the northern boundary of the Ashton Arch. The fault zone has a maximum downthrow of at least 900 feet on the northeast side. The throw decreases toward its eastern end, and a scissors effect causes the southwest end of the fault to be downthrown a little more than a hundred feet in western Will County. Movements along the Sandwich Fault Zone were post-Silurian, pre-Pleistocene. No rocks representing the intervening time are present in the area. However, major movements along the fault zone may have occurred at about the same times that the La Salle Anticlinal Belt was uplifted-in post-Mississippian, pre-Pennsylvanian time and again in post-Pennsylvanian times.

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Ref. H. B. Willman, and J. N. Payne, Geology and Mineral Resources of the Marseilles, Ottawa, and Streator Quandrangles: Illinois Geol. Survey Bull. 66, 1942.

H. B. Willman, and J. S. Templeton, Cambrian and Lower Ordovician Exposures In Northern Illinois: Trans. of Illinois Acad. Sci. v. 44, p. 109-125, 1951.

LASALLE ANTICLINAL BELT

The La Salle Anticlinal Belt is an extensive asymmetrical fold that extends in Illinois from Lee County in the northwest to Lawrence County in the southeast. The west limb dips sharply into the deeper part of the Illinois Basin, whereas the east limb dips gently into the eastern shelf area of the basin. The crest of the anticline plunges to the south-southeast. Initial deformation along the La Salle Anticlinal Belt took place in post-Mississippian time. Deformation continued through early Pennsylvanian time, particularly at the southern part of the structure. Renewed activity occurred after Pennsylvanian time, probably at the close of the Paleozoic Era.

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Refs. G. H. Cady, The Structure of the La Salle Anticline: Illinois Geol. Survey Bull. 36, p. 171-177, 1920.

J. N. Payne, The Age of the La Salle Anticline: Trans Illinois Acad. Sci. Vol. 32, No. 2, p. 5-7, 1939.

CENTRALIA FAULT

The Centralia Fault strikes nearly north-south parallel to, and one mile east of the DuQuoin Monocline in Marion and Jefferson Counties. It is a zone of several parallel faults. Net displacement is downward to the west, with maximum displacement of about 200 feet. The faults can be seen in several coal mines in the Centralia area, but they are not visible at the land surface. The faults appear to be the results of shearing stresses formed after folding took place on the DuQuoin Monocline. Relief of the stresses was upward on the east side, opposed to the east dip of the monocline. The faulting occurred in post-Pennsylvanian, pre-Pleistocene time.

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Ref. R. L. Brownfield, Structural History of the Centralia Area: Illinois Geol. Survey Rept. Inv. 172, 1954.

ROUGH CREEK LINEAMENT

The Rough Creek Lineament is a series of faults and fault zones extending generally east-west through western Kentucky and southern Illinois. In Kentucky, it includes the Rough Creek Fault Zone. In Illinois, it includes the east-west portion of the Shawneetown Fault Zone to the east and the Cottage Grove Fault System to the west. Heyl (1972, p. 885) suggests that strike-slip faulting or wrench faulting is a major component in the Rough Creek Lineament. He tentatively includes it in a line or zone of faults, monoclines, and igneous intrusions. The line extends east-west for 800 miles along the 38th parallel from West Virginia to at least as far west as the Ozark Uplift.

In Illinois, the lineament includes numerous high angle reverse faults with the south side upthrown. They appear to be the result of compressional forces from the south, and they display evidence of some horizontal movements. The eastern part of the lineament, the Shawneetown Fault Zone, is dominated by high angle thrust faulting. Displacement is locally as great as 3400 feet and may be considerably more. The Shawneetown extends westward along the prominent hills in southern Gallatin County, curves southward around Cave Hill in Saline County, leaves the Rough Creek Lineament and joins the southwest-trending Herod Fault to form the Lusk Creek Fault Zone. The western portion of the lineament, the Cottage Grove Fault System, appears to have formed at roughly the same time as the Shawneetown, but displacements are much diminished, with maximum

ROUGH CREEK LINEAMENT (Continued)

displacements of about 250 feet. From all available evidence it appears that the age of faulting along the Rough Creek Lineament is chiefly post-Pennsylvanian, pre-late Cretaceous, although some workers have suggested the possibility of later movements because of recent seismic activity in the general area.

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Ref. A. V. Heyl, The 38th Parallel Linement and Its Relationship to Ore Deposits: Economic Geology, Vol. 67, No. 7 pp. 879-894. 1972.

WABASH VALLEY FAULT SYSTEM

The Wabash Valley Fault System is a series of generally parallel faults that trends northeastward from the Rough Creek Lineament in Gallatin County, roughly paralleling the Wabash River in Illinois and Indiana to near Mt. Carmel, Wabash County, Illinois. The faults are high angle, normal faults. The faults have been observed in mines, boreholes, and surface exposures. Maximum known displacement on the faults is a little over 400 feet, although displacement of a few to 200 feet are more common. The throw of the faults appears to be the same in Mississipian and Pennsylvanian strata,and they are clearly post-Pennsylvanian in age. No displacement has been recognized in Pleistocene deposits, thus the faulting appears to have occurred in pre-Pleistocene time.

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Refs. J. A. Harrison, Subsurface Geology and Coal Resources of the Pennsylvania System in White County, Illinois: Illinois Geol. Survey Rept. Inv. 153, 1951.

D. H. Swann, Waltersburg Sandstone Oil Pools of Lower Wabash Area, Illinois and Indiana: Bull. Amer. Assoc. Pet. Geologists, Vol. 35, No. 12, 1951.

STE. GENEVIEVE FAULT SYSTEM

The Ste. Genevieve Fault System extends northwestward across Union County, Illinois, crossing the Mississippi River just north of Grand Tower. It continues in that direction through Perry County, Missouri, and then swings westward through Ste. Genevieve County. Although the system includes numerous horsts and grabens, the net displacement is down to the north and east. Maximum displacement is more than 1000 feet and may approach 2000 feet. The fault system forms a sharp boundary, a few miles wide, between the Illinois Basin and the Ozark Uplift. The faults are high angle faults with some reverse and some lateral movements. Compression from the southwest was probably an important factor in their formation. Movements along the faults occurred several times during late Paleozoic times. Without question, there was post-Mississippian, pre-Pennsylvanian movement followed by post-Pennsylvania movement. Unusually thick sections of Devonian strata are preserved in grabens of the fault system, and these may be explained by earlier faulting in perhaps Late Devonian time.

The extension of the Ste. Genevieve Fault System into Illinois has been called the Rattlesnake Ferry Fault. Presently it is called the Ste. Genevieve Fault Zone.

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STE. GENEVIEVE FAULT SYSTEM (Continued)

Refs. S. St. Clair and S. Weller, Geology of Ste. Genevieve County, Missouri: Missouri Bur. Geology and Mines, Ser. 2, V. 22, 1928.

W. F. Meents and D. F. Swann, Grand Tower Limestone (Devonian) of Southern Illinois: Illinois Geol. Survey Circ. 389, 1965.

CAP AU GRES FAULTED FLEXURE

The Cap Au Gres flexure is a sharp monoclinal fold that extends east-south-eastward through Lincoln County, Missouri, then generally eastward through southern Calhoun and Jersey Counties, Illinois. The rocks dip steeply on the southern flank of the structure, and the maximum amount of structural relief is 1000 to 1200 feet. Along much of the length of the flexure the rocks are broken by faults that trend parallel to the strike of the rocks. The faults generally are down thrown to the south and have displacements from a few to a few hundred feet. Limited exposures in the area make it difficult to determine the extent and continuity of the faults. Major deformation along the Cap Au Gres Flexure took place in post-Middle Mississippian, pre-Pennsylvanian time. Pennsylvanian strata south of the flexure are considerably lower than outliers of similar strata north of the Flexure. In addition, the Calhoun peneplain bevels across the edges of tilted Pennsylvanian strata in the area, thus indicating post-Pennsylvanian movement along the Cap Au Gres Faulted Flexure. Ruby (1952, pp. 64, 145, 146) argues that even later movement is indicated by displacement along the structure of the Calhoun peneplain and the Grover Gravel, which immediately overlies the peneplain. This displacement occurred after deposition of the gravel (probably in Pliocene time) and amounts to a little more than 100 feet. No evidence has been found to indicate any deformation of Pleistocene deposits in the area.

A pair of northwest-trending anticlines end abruptly against the flexure; they are the Dupo-Waterloo Anticline to the south and the Lincoln Fold to the North. Both anticlines have their steeper flanks to the west, and they appear to have similar geologic histories. The crests of the anticlines are offset about 30 miles. It is possible that the Cap Au Gres Faulted Flexure is a left-lateral fault in which the horizontal movement of about 30 miles offset the Lincoln Fold from its southern continuation, the Dupo-Waterloo Anticline.

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Ref. W. W. Ruby, Geology and Mineral Resources of the Hardin and Brussels Quadrangles (in Illinois): U.S. Geol. Survey Prof. Paper 218, 1952.